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MAY 1958
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JOURNAL
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AMERICAN CONCRETE INSTITUTE

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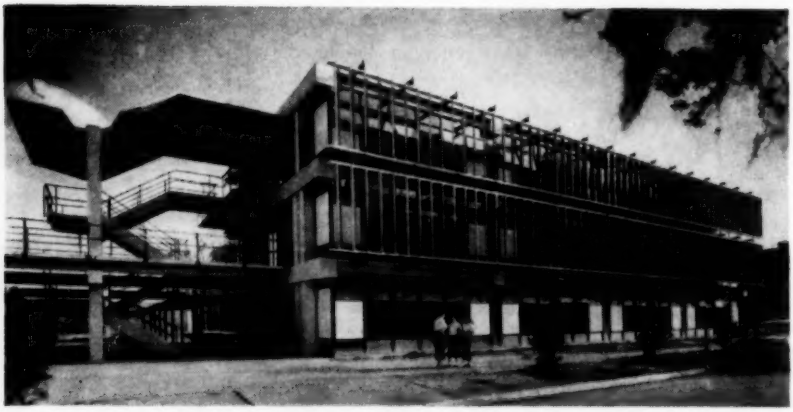

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Coming next month in the JOURNAL

JOURNAL line-up for June includes the following technical papers:

"Long-Time Study of Cement Performance in Concrete. Chapter 11—Report on the Condition of Three Test Pavements After 15 Years of Service," by F. H. JACKSON

"Static and Fatigue Strength in Shear of Beams with Tensile Reinforcement," by T. S. CHANG and CLYDE E. KESLER

"Design and Control of Lightweight Aggregate Concrete for the World Trade Center in the Ferry Building, San Francisco," by PAUL J. FLUSS

"The Effect of Temperature on Concrete Strength," by PAUL KLIEGER

"Precast Reinforced Concrete Slab Bridges with Stiffened Edges," by A. GALLIA

"Design of Concrete Mixtures Using Chicago Fly Ash as One of the Ingredients," by C. E. LOVEWELL and GEORGE W. WASHA

"Military Personnel Records Center, St. Louis, Mo.," by EARL B. COHN and W. A. WALL

"Creep and Creep-Recovery of Concrete Under High Compressive Stress," by A. M. FREUDENTHAL and F. ROLL

●

Discussion of papers published in the October, November, and December, 1957, issues will appear in the June JOURNAL, in accordance with the Institute's quarterly publication schedule for *Proceedings* discussion.

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P. O. BOX 4754, REDFORD STATION, DETROIT 19, MICHIGAN

Tentative Recommendations for Thin-Section Reinforced Precast Concrete Construction*

Reported by ACI Committee 324

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SYNOPSIS

Intended as a supplement to ACI 318-56, this report highlights design and construction practices peculiar to thin-section reinforced concrete elements. High-early-strength concrete or accelerated curing is advised. Special grading limits for coarse aggregate are suggested, and specifications for steel and admixtures are noted. Concrete of 3750 psi strength is recommended for protected locations not in contact with the ground; 5000-psi concrete is recommended for other locations. Limits for air content, W/C ratio, and cement content are proposed. Report suggests allowable design stresses, and emphasizes accurate placing of and sufficient cover for reinforcement.

Fabrication is covered in provisions on mixing, molds, casting, curing, surface treatment, and tolerances of individual elements. Supervision and inspection during fabrication are stressed, with some suggested standards of acceptance. Method and sequence of erection are also treated, including connection devices, assembly tolerances, and waterproofing of joints.

INTRODUCTION

During the past decade, there has been a gradual increase in the use of thin-section reinforced precast concrete elements in the building industry. In some structures these elements have been utilized as local framing members, such as floor, roof, and wall panels; in others, they have been adapted to provide complete structural framings.

As may be inferred from the designation, thin-section precast elements are characterized by their unusually slender proportions. They differ from conventionally cast members not only in sectional thicknesses, but also in protective cover and spacing of reinforcing bars. Because of these differentiations,

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This report in form and substance as here submitted was approved unanimously by the committee as listed above.

and in the absence of an applicable code, difficulties have been experienced in some localities with their use.

Thin-section precasting is performed by rigidly controlled operations comparable to planned factory production. The quality control thus obtained makes it possible to utilize reinforced concrete in this most advantageous form. This report is intended to serve as a guide for the engineer and the fabricator, as well as an aid to facilitate the acceptance of this method of construction.

SCOPE

This report covers the use of thin-section precast concrete elements which combine to form a complete structure or a part thereof, and may be used as a supplement to "Building Code Requirements for Reinforced Concrete (ACI 318-56)." Thin-section elements are those which have a thin skin or slab which may be stiffened by a system of ribs, as distinguished from conventional precast construction where thicker slabs and solid-section elements are used.

Precast elements may be formed by casting concrete in wood, metal, concrete, or other suitable rigid forms, or by the pneumatic application of concrete on specially prepared surfaces.

The design of precast concrete elements should be in accordance with accepted reinforced concrete theory and should conform to ACI 318-56, except as indicated herein. Thrusts, moments, and shears at various sections of structural assemblies should be computed by the application of the elastic theory using the moments of inertia of the gross concrete sections of the reinforced elements.

MATERIALS

Cement

Portland cement should conform to "Standard Specifications for Portland Cement" (ASTM C 150) or "Standard Specifications for Air-Entraining Portland Cement" (ASTM C 175). High-early-strength concrete as produced with Type III or IIIA portland cement or with Type I portland cement and accelerated curing is recommended.

Aggregate

Concrete aggregates should conform to "Standard Specifications for Concrete Aggregate" (ASTM C 33), except for the gradation of coarse aggregates which should conform to Table 1.

TABLE 1—GRADATION OF COARSE AGGREGATES

Maximum size, in.	Percent passing standard laboratory sieve, by dry weight									
	1 in.	¾ in.	½ in.	⅜ in.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100
¼	100	100	100	100	90–100	70–90	45–75	25–55	10–30	2–10
⅜	100	100	100	85–100	0–60	0–10	0	0	0	0
½	100	100	90–100	40–70	0–15	0–5	0	0	0	0
¾	100	90–100	20–35	5–10	0	0	0	0	0	0

Lightweight aggregates conforming to ASTM C 330 may be used provided they meet the requirements for strength, permeability, and shrinkage.

Maximum size of aggregate for a precast element should be no larger than one-third of the narrowest dimension between the sides of the forms of the member in which the element is cast; nor larger than:

(a) Two-thirds of the minimum clear space between parallel reinforcing bars, or

(b) Two-thirds of the minimum clear space between reinforcing bars or web reinforcement and the faces of the form.

Steel

Reinforcing steel should be intermediate-grade billet-steel concrete reinforcement bars (ASTM A 15) or rail steel concrete reinforcement bars (ASTM A 16) or cold drawn steel wire for concrete reinforcement (ASTM A 82). Deformations on reinforcing bars in precast elements should conform to "Standard Specifications for Minimum Requirements for the Deformation of Deformed Steel Bars for Concrete Reinforcement" (ASTM A 305). In wide, thin elements having faces 3 in. or less in thickness the use of wire mesh reinforcing, preferably of 2-in. spacing, is recommended. Wire mesh should conform to ASTM A 185, except that galvanized fabric is acceptable.

Admixtures

Air-entraining agents should be materials accepted for use in concrete under the provisions of "Tentative Specifications for Air-Entraining Admixtures for Concrete" (ASTM C 260). When used, calcium chloride should meet the requirements of "Standard Specifications for Calcium Chloride" (ASTM D 98) and should be limited to not more than 2 percent by weight of the cement. Calcium chloride should be dissolved in a portion of the mixing water before batching. The use of other admixtures will be limited to those which do not alter the intent of any section of these requirements, and their use should be in accordance with pertinent information in "Admixtures for Concrete" reported by ACI Committee 212, ACI JOURNAL, Oct. 1954, *Proc.* V. 51, pp. 113-148.

Quality of concrete

Concrete for precast elements to be used in locations protected from the weather or moisture and not to be in contact with the ground should have compressive strength of not less than 3750 psi at 28 days. For elements at other locations, the compressive strength should be not less than 5000 psi at 28 days.

Where air-entrained concrete is specified, the amount of entrained air should be as follows:

Maximum size of aggregate	Percent of entrained air
¾ in.	6 ± 1½
½ in.	7 ± 1½
⅜ in.	8 ± 1½
¼ in.	9 ± 1½

TABLE 2—WATER-CEMENT RATIO AND MINIMUM CEMENT IN CONCRETE

Class of concrete (28-day strength, psi, of 6x12-in. cylinders)	Maximum size aggregate, in.	Normal concrete			Air-entrained concrete		
		Maximum water, gal. per sack cement	Minimum cement, sacks per cu yd	Fine aggregate range, percent of total aggregate by weight	Maximum water, gal. per sack cement	Minimum cement, sacks per cu yd	Fine aggregate range, percent of total aggregate by weight
3750	¼	6.5	7.0		5.75	7.4	
3750	⅜	6.5	6.4	46–54	5.75	6.75	42–50
3750	½	6.5	6.0	43–51	5.75	6.3	39–47
3750	¾	6.5	5.6	40–48	5.75	5.9	36–44
5000	¼	5	9.0		4.5	9.75	
5000	⅜	5	8.25	40–48	4.5	8.75	36–44
5000	½	5	7.5	38–46	4.5	8.0	34–42
5000	¾	5	7.0	36–44	4.5	7.5	32–40

The two classes of concrete (3750 psi and 5000 psi) should conform to the requirements for maximum water-cement ratio and minimum cement content as given in Table 2.

The specified strength of the concrete in compression, as well as needed flexural strength for handling should be verified by test. Compressive strength tests should be made in accordance with "Standard Method of Making and Curing Compression and Flexure Test Specimens in the Field" (ASTM C 31) and "Standard Method of Test for Compressive Strength of Molded Concrete Cylinders" (ASTM C 39). Flexure strength tests should be made in accordance with "Standard Method of Test for Flexure Strength of Concrete" (ASTM C 78). Test results should be evaluated in accordance with ACI 214-57. In general, three samples should be taken for each normal day's casting.

ALLOWABLE UNIT STRESSES IN CONCRETE AND REINFORCEMENT

The allowable design stresses in the concrete should conform to the requirements set forth in ACI 318-56.

The tensile stresses in the concrete during handling, computed on the basis of an uncracked section, should be less than 75 percent of the modulus of rupture of the plain concrete at that time. Modulus of rupture of concrete at various ages should be determined by standard test procedures as outlined above under "quality of concrete," and related to the compressive strengths at the corresponding ages.

The allowable tensile stresses in the steel should be 60 percent of the minimum yield value, except that under temporary loading conditions during erection the unit tensile stress may be increased to 80 percent of the minimum yield strength. Allowable compressive stresses in the steel should conform to the requirements of ACI 318-56.

REINFORCEMENT

Placing

Reinforcing bars or mesh should be accurately placed and adequately secured in position by concrete or metal chairs and spacers or other approved

methods. Where the reinforcement comprises wire mesh and reinforcing bars, preassembly of such reinforcement into cages by a template or jig before placing in the forms is recommended.

The minimum clear distance between parallel bars should be not less than 1½ times the maximum size of the coarse aggregate.

Protective cover

For surfaces exposed to weather, ground, or in contact with water: Main reinforcing in beams, girders, and columns should be protected with concrete not less than ½ in. thick. Reinforcing in slabs and secondary reinforcing in beams, girders, and columns should be protected with concrete not less than ⅜ in. thick.

For surfaces not exposed to weather: Main reinforcing in beams, girders, and columns should be protected with concrete not less than ⅜ in. thick. Reinforcing in slabs and secondary reinforcing in beams, girders, and columns should be protected with concrete not less than ¼ in. thick.

FABRICATION

Mixing

Concrete should have the workability necessary so that placing and consolidating in the forms or molds can be readily accomplished without causing segregation, honeycomb, or rock pockets.

Molds

Molds should conform to the shape, lines, and dimensions of the precast elements as called for on the plans. Molds should be of concrete, steel, wood, plastics, or other suitable materials. They should be sufficiently tight to prevent the leakage of mortar, and constructed so that after the placing of concrete the tolerances indicated hereafter are maintained.

Casting

Insofar as it is practicable, the casting procedure should be such that thin sections are cast in the horizontal position with open-top forms to facilitate placing and compacting the concrete. The finished castings should be free of honeycomb or rock pockets. Vibration of rib sections is recommended.

Adequate equipment should be provided for heating the concrete materials and protecting the precast elements during freezing or near-freezing weather, in accordance with "Recommended Practice for Winter Concreting (ACI 604-56)."

Rib panels to be used in roof and floor framing should be cast with an adequate camber to obviate sagging deflections under service loading.

Curing

Concrete should be kept moist during the curing period. Precast elements made with Type I or IA portland cement should be maintained above 70 F for at least 3 days. Elements made with Type III or IIIA portland cement

should be maintained above 70 F for at least 2 days. For each increment of 5 deg below 70 F in the average curing temperature, these periods should be increased by one-half day for elements made of Type I or IA portland cement and by one-fourth day for units made of Type III or IIIA portland cement. The average curing temperature should in no case be less than 50 F.

When high-pressure steam, steam vapor, vacuum, or other accepted processes are used to accelerate curing, they should produce strength and durability properties at least equivalent to those obtained by the methods indicated in the preceding paragraph.

Surface treatment

When scoring and/or rough finishes are used for architectural treatment, the effective cover over the reinforcement should be not less than the minimum indicated on p. 925.

No aggregates or pigments for coloring purposes should be added to the concrete mix which may produce harmful chemical or physical effects to the properties of the concrete.

Where a smooth finish is required, pneumatically applied mortar may be used as a surface treatment.

Tolerances of individual elements

Precast concrete elements should be true to size and dimensions shown on the plans within the following limits at the time they are placed in the structure:

Over-all dimensions of members

± 1/16 in. per 10 ft, but not to exceed + 1/8 in. or − 1/4 in.

Cross-sectional dimensions

Sections less than 3 in.	± 1/16 in.
Sections over 3 in. and less than 18 in.	± 1/8 in.
Sections over 18 in.	± 1/4 in.

Deviation from straight line in long sections

Not more than 1/4 in. per 20 ft

Supervision and inspection

Supervision shall conform to the requirements of Section 204 of ACI 318-56.

Unless otherwise stated in job specifications, the following should govern the acceptability of castings which have developed cracks during construction:

1. Where appearance is not an important consideration, precast concrete framing members containing hair cracks may be accepted. (A hair crack is defined as a surface crack of minute width, visible but not measurable by ordinary means.)

2. Where appearance is not an important consideration, and reinforcement is given a protective coating such as cement slurry before being placed in the molds, precast concrete framing members containing cleavage cracks may be accepted. (A cleavage crack is defined as a through crack not over 0.01 in. wide, but which in the judgment of the inspector penetrates at least to the reinforcing steel.)

Proper inspection should be maintained to assure the following:

1. That preassemblies of reinforcing steel are not forced into the molds or forms because of dimensional errors in fabrication.
2. That allowable clearances and tolerances are closely followed.
3. That the position of inserts, notches, bolt holes, and connection devices in the elements during casting operations are accurately controlled.
4. That where drilling is required, as described in "connection devices" below, no serious spalling or cracking as defined in the previous paragraph (p. 926) results.

Where strength tests are required for separate elements or assemblies, the test set-up should simulate the stress conditions anticipated in the actual structure. For test purposes the element or assembly should be subjected to the total design dead load plus twice the design live load. If the tested element or assembly does not show any visible signs of distress in the form of deflection, cleavage cracks, or joint separations, it should be considered to have passed the test.

ERECTION

Methods and sequence

Precast concrete elements should be kept in their casting position until the requirements of strength recommended on p. 924 have been satisfied. To minimize strains during lifting of castings from the molds, the use of vacuum-suction lifting devices is recommended.

All elements should be handled and placed as indicated on the plans or as proposed by the contractor and approved by the engineer.

During erection, temporary loads in excess of the design load may be permitted, provided the resultant stresses do not exceed those recommended on p. 924.

Connection devices

Welding of reinforcing steel, metal inserts, and connections should be in accordance with the recommended practice of the American Welding Society. Care should be taken during welding of inserts and splices to minimize the effect of welding heat and the acceptability of any resulting cracking should be governed by the requirements on p. 926.

Drilling into or through concrete sections, by mechanical devices, for the purposes of rectifying casting errors, or to accommodate any type of connection devices or hangers, should not be allowed in sections of less than 1½ in. thickness.

Where grout or concrete is used to embed connection devices between elements or to fill allowable casting tolerances, the requirements of Section 406(a) of ACI 318-56 should be followed.

Assembly tolerances

All load bearing surfaces at columns and wall panels should be given a layer of cement grout to assure an even bearing over the entire contact surface.

Correction of minor misfits and/or any casting errors which prevent the proper assembly and fitting of elements should be reported to the engineer so that he may either correct the error or approve the method of correction to be used. Elements should not be forced or distorted to fit into position or alignment.

Weatherproofing of joints

Joints between elements may be filled with a mastic joint filler or caulking compound. Compounds meeting the requirements of federal specification TT-C-598 have been used. However, better results have been obtained with some of the polysulfide synthetic rubber caulking compounds of the type used by the aircraft industry to seal fuel tanks and aircraft cabins.

Discussion of this report should reach ACI headquarters in triplicate by Aug. 1, 1958, for publication in the Part 2, December 1958 JOURNAL.

Title No. 54-52

High Strength Steel and Concrete Result in Minimum Column Sizes*

By FRANK W. CHAPPELL†

SYNOPSIS

Architectural design of 265 ft high building imposed severe structural restrictions which were met by using a lightweight structural concrete frame, concrete strengths as high as 6000 psi, and alloy steel bars with yield point of 82,365 psi in columns of lower stories. The saving in dead load combined with unusually high strengths of concrete and steel permitted the architect's clearances to be maintained. Unusual structural design features are described as well as precast concrete grills which break up the direct rays of the sun against considerable areas of windows and walls.

GENERAL DESCRIPTION OF BUILDING

The 102-unit apartment house is 24 stories high, completely air-conditioned, and provided with an adjoining garage having an ultimate capacity of 270 automobiles. Exterior terraces for all apartments, and outdoor swimming pool with cabanas are included in the plan. Total estimated cost is $5,000,000. Color contrast in the building façade was achieved by varying the aggregates used in the structural concrete and in the precast grills which face the building. (Another photograph of the nearly-finished building appears on this month's News Letter cover.)

Fig. 1 shows a simplified framing plan for a typical floor. The central main core is 77 ft 4 in. x 101 ft 4 in., with wings projecting from each of the four corners. The floor plan was so worked out that all of the five apartments on each of 20 floors open on a central service shaft containing elevator, stair, and incinerator enclosures. To add to the problems of structural design, each apartment is provided with an exterior terrace or porch. Inverted spandrel beams were not continued across the terraces and lateral bracing was made difficult. Therefore, the central enclosure walls around elevators, stair halls,

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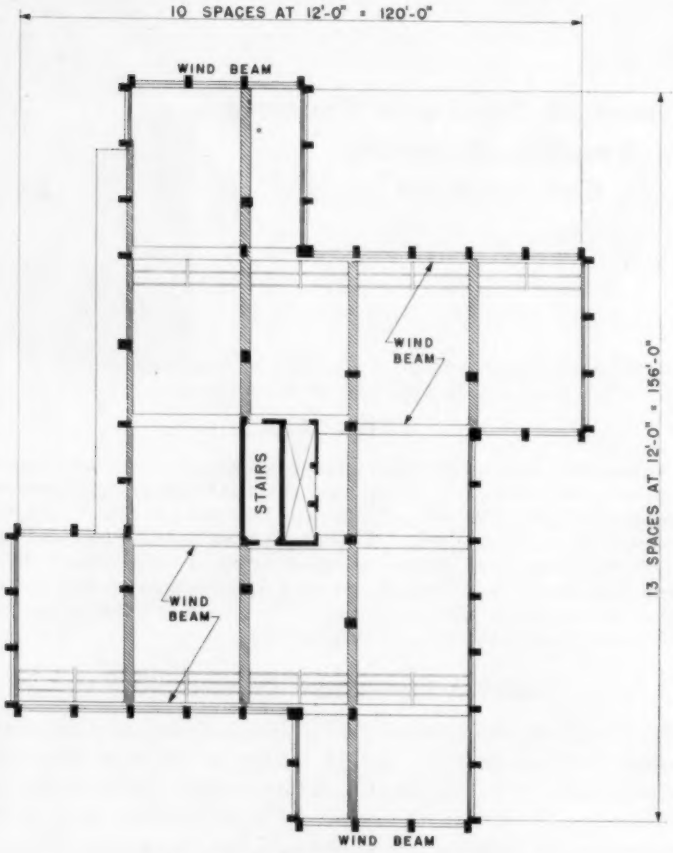

Fig. 1—Typical floor framing plan

and incinerator were made of concrete having the same strength as that for columns in any floor, and anchored into a heavy block of concrete sunk into the rock foundation.

The column spacing module for the building is 12 ft. Exterior columns are typically 16 x 32 in. in section and project 24 in. outside the faces of spandrel beams. Except at terraces, the 8 in. thick spandrel beams are inverted 28 in. above the floor line, and each has a 16-in. top flange, which serves as a window sill, and projects 8 in. beyond the face of the spandrel beam. Since the floor framing system is uniformly 16.5 in. thick the total depth of the spandrel section is 44.5 in. and is shaped like an irregular Z. A typical spandrel beam section may be seen in Fig. 2.

The floor system is simple, consisting of joists, floor beams, and wind beams, all having a uniform depth of 16.5 in. Steel "pans" 14 in. deep were used

throughout, with 2.5 in. cover slab. Joists and east-west wind beams are parallel. Beams which support floor joists extend in a north-south direction. Most of the beams and joists span 24 ft on center lines, though there are some variations. Fitting the numerous openings required by the mechanical plans proved complicated. Every commercial width of steel form was used in casting the 115 different types of joist required for typical floor.

Normally joists are 4 in. wide at the bottom and their sides batter 1 in 12, making the top width 6⅓ in. Only one straight bar was placed in the bottom of a 4-in. joist. In the top one #4 bar was run continuously to act as mesh support and additional straight top steel placed as required by bending moment. Beams received essentially similar treatment in that all have continuous top steel, the areas being varied to meet anticipated stresses. There are no bends, other than hooks, in any beam or joist bars. Fig. 3 shows typical floor framing details, looking upward at the junction of a column with beams and joists.

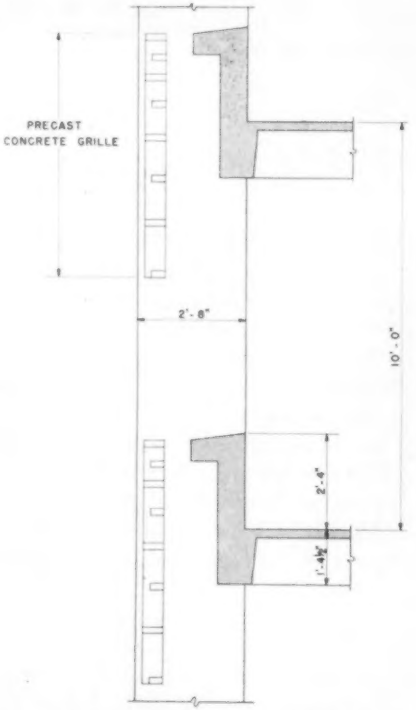

Fig. 2—Typical wall section showing Z-shape of spandrel beam

COLUMN DESIGN

Exterior columns, typically 16 x 32 in., posed a problem for the designers. Some columns were of necessity designed to take bending stresses due to wind loading, as well as direct stresses, and would have required as many as 21 #11 bars of hard grade carbon steel, using a nominal allowable stress of 20,000 psi. Had spirals been used, three sets would have been required, making a heavy and complicated assembly, with vertical steel not advantageously arranged to resist bending. The shape of the columns and their projection from the floor system made the use of structural steel difficult and perhaps impractical. Also, at the time the design was made, delivery promises on steel shapes were far in the future. Add the desire of the owners for a concrete exterior, and it was decided to use tied steel in columns. But to have tied 21 bars in a 16 x 32-in. column in accordance with the ACI Building Code would not have left spaces large enough to permit the use of a vibrator. This was the problem.

An attempt was made to procure #18 bars of high carbon steel having a yield point not lower than 50,000 psi. This size, with a cross sectional area of

TABLE 1—PROPERTIES OF ALLOY STEEL

Chemical analysis, percent	Average
Carbon	0.28–0.33
Manganese	0.80–1.00
Silicon	0.35 maximum
Nickel	0.40–0.60
Molybdenum	0.20–0.30
Chromium	0.40–0.60

Physical properties (Average of six heats)	
Yield point	82,365 psi
Ultimate strength	101,436 psi
Elongation in 8 in.	11.36 percent

4.0 sq in., is not stocked by Dallas fabricators. The mill would not roll the bar without an order large enough to take the output of a set of rolls for a full day's run. This project could not economically use such a quantity and the use of #18 bars was abandoned.

A limited quantity of #11 bars of alloy steel sufficient to reinforce all columns to the sixth floor was finally secured. This actually means seven stories of columns, since the building has a basement and a mezzanine floor. The steel is classified by the mill as SAE 9400-modified. Its physical and chemical properties are shown in Table 1.

Columns that would require 21 #11 bars of hard grade steel needed only 14 of alloy steel, since the nominal unit stress is increased from 20,000 to 30,000 psi. This made for easy erection of steel and placement of concrete. Working plans

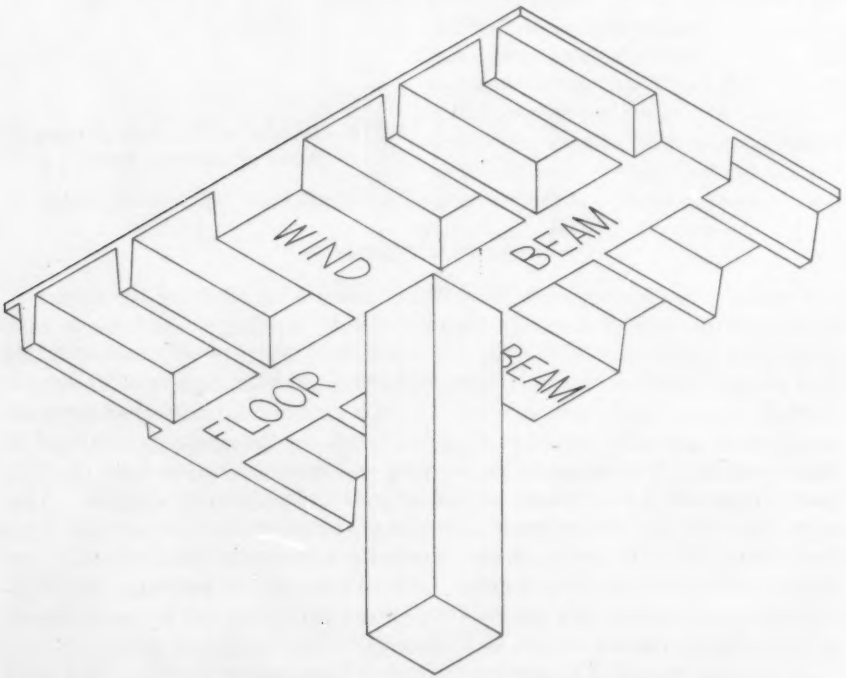

Fig. 3—Details of typical floor joists and beams

detailed the arrangement of bars and ties in all columns. Fig. 4 shows how maximum reinforcement of 14 #11 alloy steel bars was disposed in a 16 x 32-in. column.

Fig. 4—Column reinforcement details

Full advantage of the high yield point could not be used, since the maximum nominal stress is limited by the ACI Building Code to 30,000 psi. But even with this restriction, the use of alloy steel represented a saving over the lower priced carbon bars as shown in Table 2.

The rolling mill limited the size of this order to 115 tons because of prior commitments. In order to determine the total amount of the saving possible from the use of high yield point column reinforcement it is necessary to assume that sufficient #11 bars of 50,000 psi yield point steel could have been placed in all columns. If this could have been done an additional 57.5 tons would have been required at a cost of $5083. Add to this the cost of placing 57.5 tons at $42.50 per ton; the total saving would have been $7527 on 115 tons of alloy steel, or $65.45 per ton.

Welded sleeve splices

Even with high yield point steel some of the 16 x 32-in. columns did require 14 #11 bars. The difficulty of lapping and tying so many bars in so narrow a column led to an investigation of welded sleeve splices. Two samples were prepared and tested. Fig. 5 shows the assembly of the test specimen and the results of tension tests. The sleeve is of cold drawn tubing with walls 0.25 in. thick. Sectional areas of bars and tubing are practically equal. Continuous fillet welds were made with a 5/32-in. electrode, Classification 100-16. All samples failed in the welds. A third sample was tested later as a check and it likewise failed at a total tension of 85,000 lb. All far exceeded in strength any requirement of this project.

Area of the weldment is approximately 1.35 sq in. and the unit stress in the weld metal 63,000 psi at failure. Since this is less than the 75,000 psi on which the design was based, the splice apparently violates Section 1103 (c) 3 of

TABLE 2—COST COMPARISON OF ALLOY AND CARBON STEELS

Reinforcement	Working stress, psi	Cost per cwt	Cost per kip of load capacity per cwt
Carbon bars	20,000	$ 8.40	$0.420
Alloy bars	30,000	10.39	0.346
Saving credited to alloy bars			$ 0.074
Saving expressed as percentage of cost of carbon bars			17.6 percent

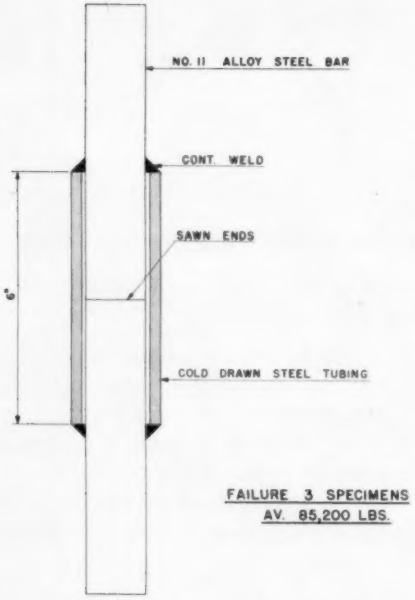

Fig. 5—Welded splice, alloy steel bars

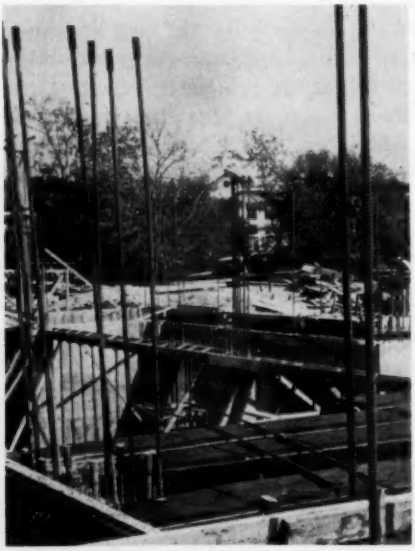

Fig. 6—Construction view of reinforcing bars

ACI 318-56 which defines an approved welded splice. It must be kept in mind, however, that the weld is not required to transmit any direct column load, but only any possible uplift caused by wind and negative bending stresses. Direct downward loads are transmitted by full section bearing.

Square bar ends

Alloy steel bars were furnished cut to exact length by sawing. Welded connections were used in wind bent columns but in all others simple pipe sleeves served to center each bar over the bar below, saving the cost of the cold drawn tubing and two welds. Fig. 6 shows some of the bars during construction. An analysis of costs shows that welded splices saved 12 cents each over lapped splices and, of course, made concrete placement easier. Where welding was not required, the simple butt bearing was much cheaper than lapped slices.

The length of a lapped splice of a #11 bar of alloy steel would have been 3.53 ft and its weight 18.75 lb.

Two-story bar lengths

The number of splices in alloy steel bars was halved by the use of bars in two-story lengths. Starting off the footings, one-half of the bars in each column were extended to 3 in. above the first floor. The other half extended to 3 in. above the mezzanine floor. Thereafter, two-story lengths were furnished to the sixth floor.

Two-story bar lengths proved quite practical for this size bar. Perhaps they would not stand erect in smaller sizes. But they saved the cost of many splices and provided full continuity for half of the bars. No diffi-

TABLE 3—PROPORTIONS, WEIGHTS, AND QUANTITIES OF LIGHTWEIGHT CONCRETE, APARTMENT BUILDING ONLY

Strength at 42 days, psi	Proportions					Quantity		Unit weight, lb per cu ft	Total weight, kips
	Cement, lb	Lightweight aggregate, lb		Sand		cu yd	cu ft		
		Fine	Coarse	lb	Percent*				
3750	541	533	694	545	21.4	9,397	253,719	94.1	23,875
5000	705	506	658	516	19.7	628	16,956	97.3	1,650
6000	800	498	648	508	18.9	606	16,362	99.5	1,628
						10,631	287,037		27,153
For hard rock concrete							287,037	149	42,769
Saving in concrete dead load due to the use of lightweight concrete, kips									15,616

*Percentage by weight of sand in total weight of air-dry concrete. Sand is also 30.7 percent of dry weights of the sum of all aggregates.

culties were encountered in placing the steel and the cost of erection was no greater than for conventional lengths. The use of column steel of high yield point, having square cut ends and with splices staggered was proposed in a paper published in the ACI JOURNAL in April, 1941.*

LIGHTWEIGHT CONCRETE

Quantities and weights of lightweight concrete are given in Table 3. Even with the use of high yield point steel, limitations on column sizes demanded unusually high strength for lightweight concrete in the lower stories. All concrete was proportioned and controlled by the Southwestern Laboratories which kept an inspector on the job during all concreting. Cylinders were made from every delivery by the ready-mixed concrete trucks. All concrete attained the specified compressive strength.

The saving of nearly 8000 tons of dead load by the use of expanded shale rather than crushed rock as an aggregate was one of the factors which allowed the extraordinary design of the building to be expressed in concrete.

Further savings in weight could have been made by using no sand. (Table 3 presents the proportions of the three strengths of concrete.) However, the architect needed the sand to attain the color he desired and the concrete as proportioned and produced was light enough to serve the purpose. Basement walls and floors, footings, and columns built into basement walls were made of hard rock concrete having a compressive strength of 3750 psi.

Concrete strength

All columns below the third floor (not including those cast with basement walls) were made of lightweight concrete having a compressive strength of 6000 psi at 42 days. Columns between third and sixth floors required 5000-psi concrete. Columns above the sixth floor were made of 3750-psi concrete, as was the entire floor system for all floors. Forty-two days were allowed for the

*Germundsson, Thor, "Columns with High Yield Point Reinforcement Designed under the ACI Code," ACI JOURNAL, Apr. 1941, *Proc.* V. 37, pp. 569-576.

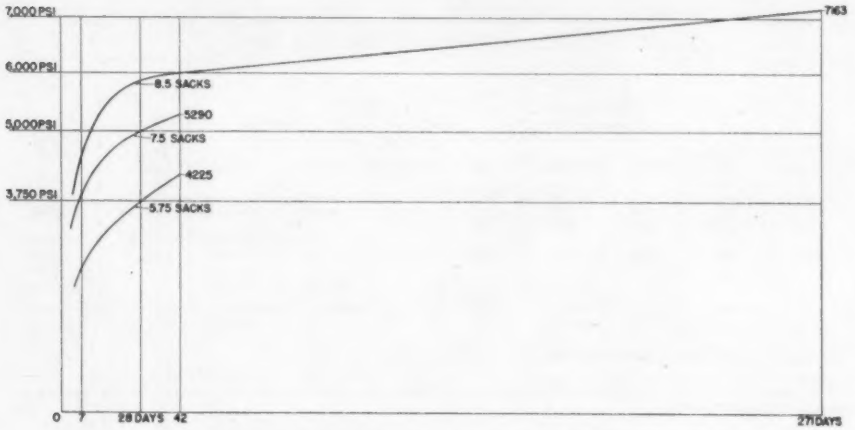

Fig. 7—Time-strength curve for concrete test cylinders

attainment of the design strength. Previous experience with concrete made of expanded shale aggregates has shown that the cylinders continue to gain strength after 42 days, in the laboratory. Some doubt exists as to whether a similar long time increase would be found if cylinders were cured under job conditions, and the limit of testing was placed at 42 days. In Fig. 7 the time-strength graphs of cylinder tests are shown.

PRECAST CONCRETE GRILLS

Factory produced concrete grills were designed for function as well as architectural wall treatment. They shade a considerable percentage of window and wall areas (Fig. 8) from the direct rays of the sun, thus reducing heat transfer, an important consideration for an air-conditioned building in a climate where summers are likely to be long and hot.

Exterior faces are in relief, since there are two surface planes. This was accomplished by making some members 6 in. deep and others 4 in., the back side being in a single plane. The grills weigh from 15 to 19 lb per sq ft and from 1000 to 2000 lb per grill. A total of 865 grill sections was required.

All steel reinforcement and hangers were galvanized. Concrete for the grills was made of expanded shale aggregate with 5.75 bags of cement per cu yd and was furnished by a ready-mixed concrete plant. The concrete is a very light gray—almost white, a different color from the structural concrete. The latter is mixed with an expanded shale aggregate processed at Dallas, which contains iron oxides and produces concrete of a pinkish shade. The grills were made with another expanded shale processed at Strawn, Tex., which produces light gray concrete. This color was further lightened by the addition of 1.5 lb of titanium oxide per sack of cement. Concrete grill on one of the terraces is shown in Fig. 9.

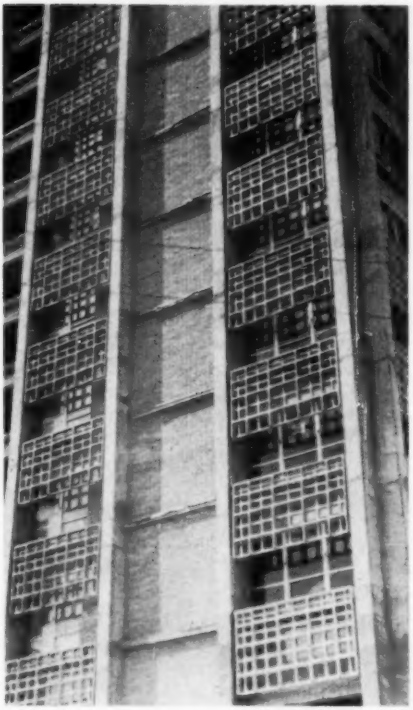

Fig. 8—Precast grills shade window and wall area

Fig. 9—Terraces on west will have grills continuous from floor to floor

Patterns for these grills were cut from 6-in. mahogany lumber. All corners and all outside face edges were rounded, making every cut compound. Polyester resin and glass fiber cloth and matting were used to make the basic mold from the wood pattern. The laminated form was strong and rigid enough to support the weight of workmen placing steel and lubricating the form itself, but had sufficient flexibility to permit stripping.

CONCLUSIONS

In tall building frames of reinforced concrete, in which limitations on column sizes and shapes are fixed by others than the structural engineer, column dimensions can be reduced by one or more of the following:

1. The use of high yield point (75,000 psi minimum) steel of satisfactory ductility for column reinforcement.
2. The use of lightweight (less than 100 lb per cu ft) concrete throughout under competent laboratory supervision.
3. The use of high strength (up to or even greater than 6000 psi) concrete where required to maintain required clearances.

ACKNOWLEDGMENTS

The author wishes to thank the following for their assistance in the design and construction of the building and in the preparation of this paper: T. A. Cullum, Jr., structural engineer in the office of the author, who performed a major part of the design and who made the drawings used herein; Howard R. Meyer, architect for the building; Edward T. Dicker and Jerome J. Frank, owners and builders of the structure; and Matt M. LaVail, president of the Morazzo Corp., manufacturers of the precast concrete grills.

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Title No. 54-53

Warping of Reinforced Concrete Due to Shrinkage*

By ALFRED L. MILLER†

SYNOPSIS

Objectionable deflections and deformations of structural elements, especially relatively thin slabs and shallow beams of buildings, usually are ascribed to faulty workmanship or materials, creep, and plastic flow when in fact they are evidence of warping. Warping due to temperature and moisture differentials is generally recognized but the effect of shrinkage has been relegated to the realm of uncertainty by attributing it to creep or plastic flow.

Experimental investigation reveals that warping of reinforced concrete members due to shrinkage during the period of seasoning is an inherent characteristic of reinforced concrete that can be anticipated and controlled. A realistic theory is developed by which the amount of warping can be predicted and provision made for its reduction or elimination.

INTRODUCTION

Volume change due to rise and fall of temperature is characteristic of all structural materials, and the volume of certain materials is affected by moisture changes. Several materials, particularly wood and concrete, inevitably undergo a period of seasoning during which the appreciable volume change can cause changes in shape that are evidenced by objectionable deformations and deflections.

Warping is alteration of shape caused by differential volume changes within a body arising from nonuniform temperature, moisture, and material properties. The process of seasoning of reinforced concrete is accompanied by shrinkage due to setting, hardening and drying, and the interaction of concrete and reinforcement with consequent effects on size and shape.

Shrinkage of plain concrete has been extensively considered. Reliable information on the numerous factors and relationships affecting magnitude of shrinkage strain of unreinforced concrete is recorded, but the interaction of concrete shrinkage and steel reinforcement has received little attention and at present is almost entirely a matter of abstract speculation. Warping due to this interaction has been recognized but has been relegated to that category of uncertainties encompassed by the subjects of creep and plastic flow.

Warping due to shrinkage was considered by J. R. Shank[1] in 1935 incidental to his notable treatment of plastic flow. The suggested formula was derived from the classical theory of stress distribution, the theory of flexure, and the elastic hypothesis. Validation by physical experiment was not indicated. Several publications and texts have quoted and illustrated the application of

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this formula. G. A. Maney[2] showed the importance of warping in connection with studies of long-time yield of loaded members. G. W. Washa[3] in 1947 reporting on the plastic flow of thin reinforced concrete slabs based on 5-year study properly considered the importance of warping as indicated by collateral auxiliary specimens thereby providing valuable basic data, analysis of which, however, was not pertinent to the report.

In recent years, many cases of dished thin slab floors, sagging shallow beams, and drooping marquees and cantilevered appendages have caused consternation and controversy among the building owners, architects, structural engineers, contractors, and suppliers of materials. Faulty design, material, and workmanship are naturally suspected but in most cases tests have shown greater strength than anticipated and satisfactory elastic behavior. It then becomes obvious that deflections which apparently indicate weakness and flexibility are warping deformations which are coincidentally similar.

Analysis of warping deflections as observed in actual structures by means of the current warping formula shows no correlation between theory and fact. Realistic interpretation of the phenomenon, a reliable means of evaluation and prediction, together with validation thereof by physical experiment are necessary to assure more satisfactory performance of thin slabs and shallow beams. Studies herein reported are part of a 5-year program of research for the purpose of obtaining accurate information concerning warping and developing a reliable means for expressing the relationships.

GENERAL CONSIDERATIONS

Lack of information regarding warping due to shrinkage and the inadequacy of the present mathematical evaluations emphasize the need for comprehensive study.

Because warping is strictly a geometrical phenomenon due to completely autogenous dimensional changes, it is apparent that conventional concepts of stress and elasticity are not determining factors. Experimental investigations verify this observation and demonstrate that strain is the single decisive factor. Inevitably, strains produce stresses but their relationship is not essential to the analysis of warping. Extensive study of the experimental results clearly demonstrates that warping is no function of f_c' and E and further references to these irrelevant items are intentionally omitted, even though a considerable portion of the experimental studies was devoted to them.

Assumption that concrete is homogeneous and isotropic is prequisite to this study. Great attention was paid to fulfillment of this requirement by careful batching, placing, handling, and storage of the test specimens. It is axiomatic that an initially straight specimen of uniform, identical material and dimensions throughout its length will be subjected to identical deformations in each element of length and tend to describe a circular arc as the result of eccentric cross-sectional influences, and that when its sections are symmetrical, no curvature will develop. This postulate was substantiated by the behavior of all specimens.

SHRINKAGE

Shrinkage is an inherent characteristic of portland cement concrete that is associated with the chemical and physical processes of setting, hardening, and drying. The amount of shrinkage is affected by type of cement, type of aggregate, proportions, methods of placing and curing, temperature, and humidity. Free shrinkage of plain concrete expressed as triaxial linear strains has maximum values in the range between 400 and 800 millionths for Type I cements and commonly used aggregates and proportions. Values of 500, 600, and 700 millionths are commonly accepted for low, medium, and high shrinkage respectively. These magnitudes are measured from the instant of placement to stabilized dimensions that will be reached in periods of 3 to 18 months under ordinary circumstances. Between 20 and 30 percent occurs in the first few days involving initial and final set and hardening. About 80 percent of the total is due to drying. Free shrinkage causes change in size but no change in shape.

Steel reinforcement restricts but does not prevent shrinkage in its effective direction. It prevents free shrinkage of the concrete immediately adjacent to the steel to which it bonds, thereby inducing tension in the concrete and compression in the steel. The amount of concrete that participates in this interaction determines the compressive effect on the steel. When this interaction is eccentric with respect to the cross section, unsymmetrical strains cause warping.

GEOMETRY OF WARPING

Warping is curvature of supposedly straight lines and plane surfaces. It may be expressed and measured in terms of either translational or rotational displacements. The former is preferable for physical description and measurements, and the latter is more suitable for mathematical purposes. The relation between the two methods of expression is derived in Fig. 1 (a).

The following symbols and units will be used as notation.

Δ = displacement, in.
L = length, in.
R = radius of curvature
k = curvature in millionths of radians per in.
ε_c = concrete strain *after bonding*, in millionths
ε_s = steel strain in millionths
d = effective depth, in.
t = total depth, in.
e = eccentricity, in.

Curvature is the result of unsymmetrical volume changes expressed in terms of linear strains. When an initially straight member becomes curved, parallel transverse sections become radial by rotational displacement. The amount of curvature can be found when the strains at any two points on the section are determined. Selecting one point on the reinforcement and the other point on the concave face of the concrete, the relationship between curvature and strains is depicted in Fig. 1 (b) for an element of length. All elements comprising the length of a straight member within which the material and sections are uniform will adopt identical curvatures and describe a circular arc.

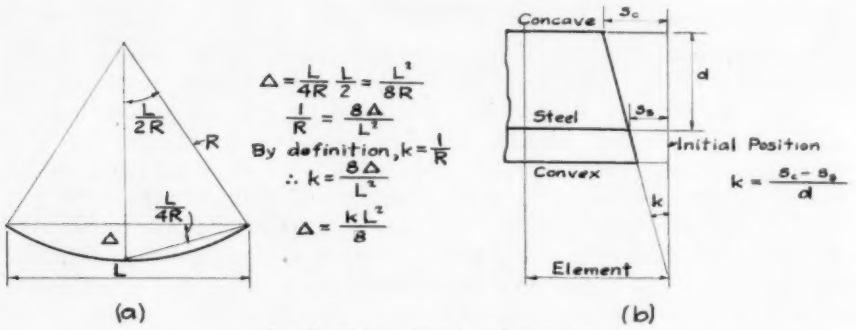

Fig. 1—The geometry of warping

The applicability of these simple abstract relationships to reality, the determination of their validity when applied to structural members, and the specific properties of the actual materials which are pertinent can be ascertained only by experimental research.

PROGRAM OF TESTS

Analysis of warping of reinforced concrete heretofore has been limited to abstract speculation. Even though its importance has been recognized, there is no record of physical research on the subject. The minute measurements required together with the time relationships and the uncertainty of factors involved presented practical problems that were resolved by numerous exploratory tests. As a result, the program which is the basis of this report was developed.

Two mixes approximating 3000 and 4000-psi commercial concrete were adopted. Regular Type I cement and good quality feldspathic aggregates were used. The 3000-psi mix contained 4.75 sacks per cu yd and 8 gal. water per sack. The 4000-psi mix contained 6.5 sacks per cu yd and 8 gal. water per sack. By careful batching fair uniformity was attained, although variations will be observed in the respective series of test specimens. Standard cylinders for each batch provided complete strength and elasticity information which, with minute details of fineness modulus, etc., are intentionally omitted from this report because they were found to be entirely irrelevant to the subject of warping.

After considerable experimentation with methods of measurement and the practical aspects of preparation, handling, and storage, a standard test specimen 42 in. long and 3.25 in. wide was selected. Each series included thicknesses of 2.5, 3, 4, 5, and 6 in. Reinforcement by one #3 bar with 0.75-in. clear coverage and by two #3 bars with 0.75-in. clear coverage is intended to represent possible behavior of thin slabs in the range of $p = 0.0063$ to $p = 0.0314$. The four series which comprised the program are hereinafter referred to as:

Series 1, two bars, 3000 psi	Series 3, one bar, 3000 psi
Series 2, two bars, 4000 psi	Series 4, one bar, 4000 psi

Series 1 and 3 are shown in Fig. 2.

Each series was made from a single batch. In addition three 6 x 12-in. cylinders, four 3 x 3 x 11-in. shrinkage specimens, and two 1.5 in. thick unreinforced specimens were prepared.

Warping was measured by a three-point gage on a length of 39 in. between metal points and a ten-thousandth in. dial gage at midpoint. Gage points on both the convex and concave faces assured accuracy of readings and checked consistency of results. The gage and the means of support to eliminate dead load deflection are shown in Fig. 3.

Free shrinkage of the four specimens of the batch of each series was measured by the device shown in Fig. 4.

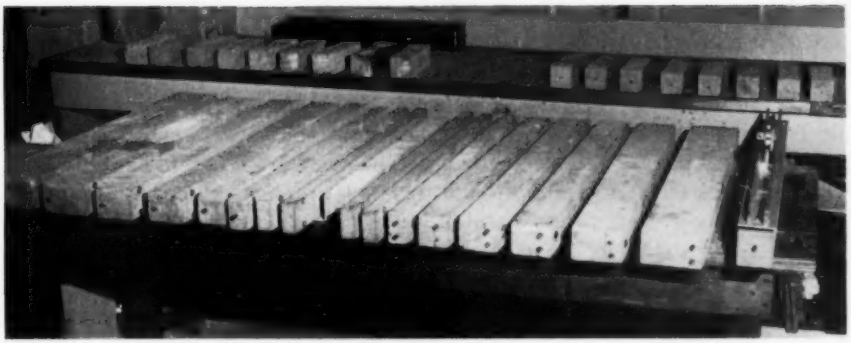

Fig. 2—Specimens of Series 1 and Series 3; shrinkage specimens in the background

All warping, control, shrinkage, and cylinder specimens of each series were made from a single batch. Forms were removed and gage points set within 24 hr at which time first readings were taken. The cylinders were cured and tested in accordance with standard specifications. All other specimens were placed in a ventilated room with a temperature that approximated 65 F with a relative humidity of about 50 percent throughout the program to approximate usual practice. Specimens were placed on their sides and alternated in position to minimize nonuniform drying and other factors that might inhibit free warping.

TEST RESULTS

Measurements of warping deflections and shrinkage strains provide the basic physical data for the analysis of the process and its evaluation. Both are time-dependent, consequently frequent readings for a long period are necessary for accurate qualitative and quantitative appraisal. For the first month, daily observations were made beginning with the first day. Thereafter, less frequent readings served to obtain an accurate picture. Series 1 and 3 were observed for a period of 280 days, Series 2 for 100 days, and Series 4 for 75 days. A period of 100 days was found sufficient for the essential basic data from which relationships and maximum values can be obtained.

All the deflections were measured to the nearest 0.0001 in. Readings were consistently regular without deviation from a smooth curve. The results for 120 days are shown in Fig. 5 for Series 1 and 2 and in Fig. 6 for Series 3 and 4. Deflections are expressed as curvature k in millionths of radians per in. for the

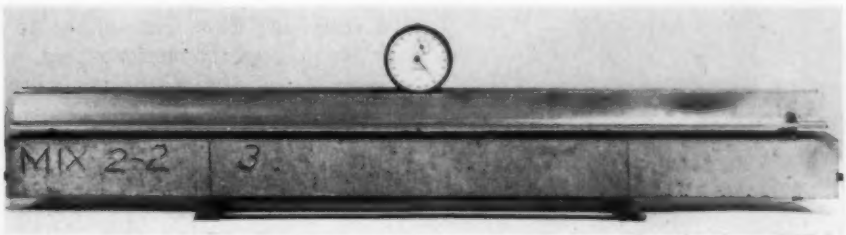

Fig. 3—Gage support to eliminate dead load deflection

3, 4, 5, and 6-in. specimens of each series. The 2.5-in. symmetrically reinforced and the 1.5-in unreinforced specimens fulfilled expectations and did not warp.

The four shrinkage specimens of each series were subject to the same conditions as the warping specimens. In spite of their presumable uniformity, identical strains did not develop and each set of four showed deviations in the range of ± 10 percent from an average. The average free shrinkage strains in millionths for each series for a period of 120 days are shown in Fig. 5 for Series 1 and 2 and in Fig. 6 for Series 3 and 4.

GENERAL OBSERVATIONS

In the course of the program, certain characteristics became evident which are important to proper evaluation of the recorded results. As generally acknowledged, exact uniformity cannot be attained in concrete specimens; natural deviations in properties can be minimized but not eliminated. This does not preclude the mathematical treatment of numerical results but does emphasize the need for realistic appraisal of both the observable facts and the theoretical analyses.

Shrinkage started during the first day and continued without interruption for 9 months or more at a diminishing rate. Warping started during the fourth or fifth day and progressed at a rate resembling shrinkage thereafter. For all series, the 5-day shrinkage is almost exactly 25 percent of the final total free shrinkage. Therefore, warping is caused by the shrinkage at a given instant minus the 5-day shrinkage. Final warping is determined by approximately 75 percent of the total free shrinkage. These observations are clearly depicted by the basic data recorded in Fig. 5 and 6.

Fig. 4—Device to measure free shrinkage

Concrete shrinkage is restricted by reinforcement within a limited range and beyond this limit free shrinkage occurs. Shrinkage strains of the concave face of a warped specimen never exceed free shrinkage and will equal free shrinkage when outside the restrictive range of the reinforcement.

Symmetrical sections do not warp regardless of the magnitude of concrete shrinkage. The combination of eccentric reinforcement and concrete shrinkage causes warping. Also, plain concrete with unsymmetrical shrinkage properties will warp.

ANALYSIS OF RESULTS

Warping as a function of shrinkage is shown by Fig. 7. Free shrinkage strains are shown as abscissas. Note that direct proportionality is apparent after $s = 250$ which corresponds to conditions at 8 to 12 days. Up to that time, the

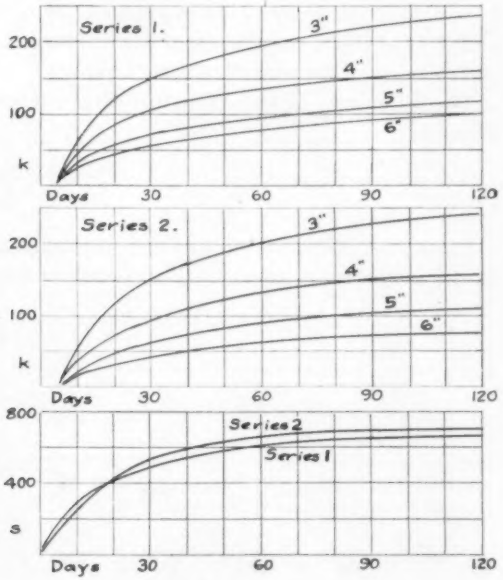

Fig. 5—Deflections expressed as curvature k for Series 1 and 2; free shrinkage strains, bottom graph

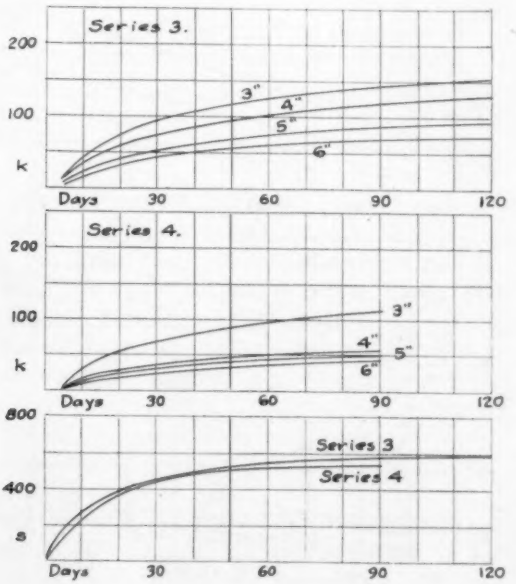

Fig. 6—Deflections expressed as curvature k for Series 3 and 4; free shrinkage strains, bottom graph

members are obviously in a state of complex internal adjustments. After the warping-shrinkage relationship is established, undoubtedly interaction between the steel and the concrete includes regular adjustments of internal stresses and maintenance of equilibrium. It is to be expected that creep and plastic flow will act simultaneously with shrinkage to the end that stress intensities will be minimized. Determination of stresses due to this interaction is not considered in this study, but supplementary tests show that they are considerably smaller than indicated by current abstract theory.

The geometry of the curvature-strain relation shown by Fig. 1 (b) is expressed:

$$k = \frac{s_c}{d}\left(1 - \frac{s_s}{s_c}\right) \quad \ldots\ldots(1)$$

Inspection of Eq. (1) shows that it is consistent with the direct proportionality of the test results shown in Fig. 7 and furthermore indicates that the ratio

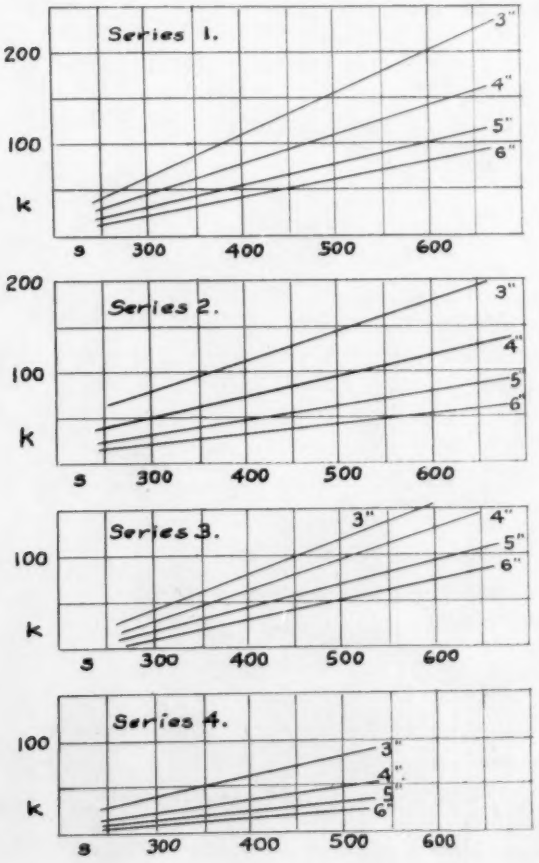

Fig. 7—Warping as a function of shrinkage

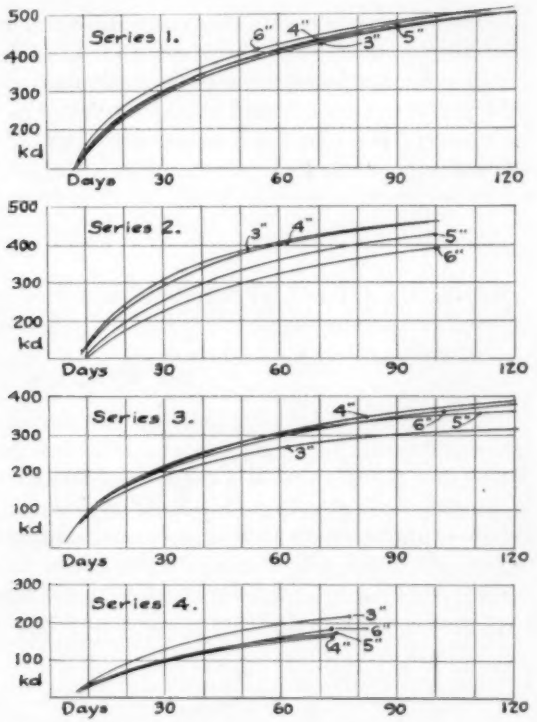

Fig. 8—Variation of kd with increasing age

s_s/s_c is a constant for each series. Validity of this appraisal is demonstrated by Fig. 8 in which values of kd are shown to be substantially identical for all specimens of each series after complete interaction is attained at approximately the tenth day. This finding indicates that s_c must be the free shrinkage strain after bonding and that s_s/s_c is determined by the coverage of the steel rather than the conventional ratio p.

Further analysis of Eq. (1) clearly indicates that it is not general since obviously it does not apply when $d = t/2$ and $k = 0$. It is applicable to members where d/t exceeds $2/3$, which is the usual case. For smaller eccentricities, where d/t is less than $2/3$, the strain on the concave face is less than s_c and curvature is controlled by the balance between the rotational influences of the concrete strains about the position of the steel. Both cases are illustrated in Fig. 9. It is apparent that transition occurs when $d/t = 2/3$.

The existence of two cases was not anticipated in the research program; consequently, study of the effects of small eccentricity is incomplete. However, supplementary tests support the validity of the following theoretical analysis and establish the range of applicability as $d/t = 2/3$.

Refer to Fig. 9 (b). By interaction with the steel, the concrete shrinkage strain is prevented from attaining the value s_c, thereby causing tension in the concrete and equal compression in the steel. Rotational deformation develops

in order to maintain rotational equilibrium. Assuming that concrete exerts tension in proportion to the amount of strain by which it is prevented from reaching its unstressed magnitude s_c, the moment on the concave side of the steel will equal the moment on the convex side. Expressing this relationship in terms of the quantities as indicated yields the following expression for curvature:

$$k = \frac{s_c}{t}\,(1 - s_s/s_c)\left(\frac{e/t}{1/12 + (e/t)^2}\right) \qquad \text{.....(2)}$$

Solving Eq. (1) and (2) simultaneously produces the result, $e/t = 1/6$ and $d/t = 2/3$, thereby fixing their ranges of applicability.

THE TERM, s_s/s_c

The relationship between shrinkage strain induced in the steel, s_s, and the free shrinkage strain of the concrete after bonding is the principal factor that determines the warping of a member of specified dimensions. Research now in progress indicates that shrinkage imposed on a given bar arises from the action of the concrete immediately surrounding the bar and that additional concrete outside a specific range has no effect. In other words, bars of all sizes surrounded by a large amount of concrete will have identical shrinkage strains. Exploratory tests indicate that when any bar is subject to shrinkage of a

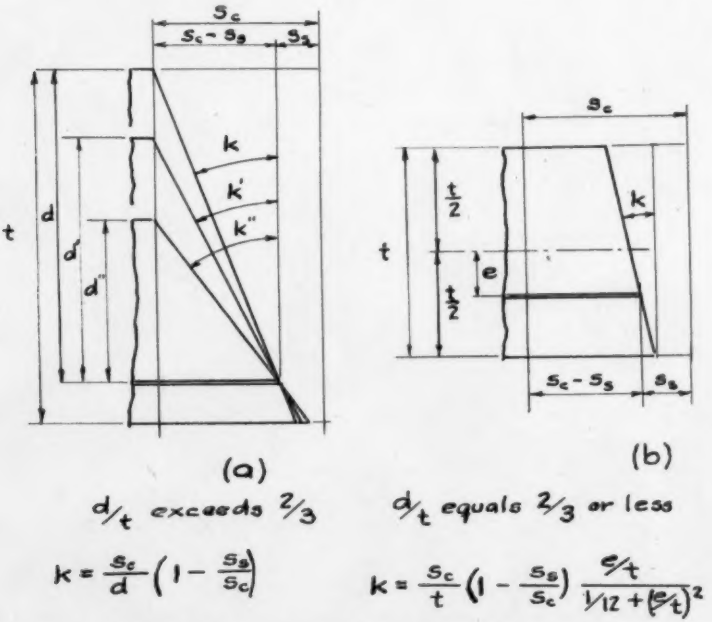

Fig. 9—Expressions for curvature for two ranges of ratio d/t

surrounding concrete area in excess of 100 A_s, ε_s approximates 0.7 ε_c. Observations indicate that an exposed bar even when bonded to adjacent concrete is negligibly affected. Therefore, it appears that coverage related to bar size controls the value of $\varepsilon_s/\varepsilon_c$ which will be in the range between 0 and 0.70. However, precise values of this ratio can not be assigned until they are ascertained by experimental investigation of this unexplored aspect of the subject.

Analysis of test results reported in Fig. 5 and 6 shows consistent values for all specimens as follows:

Series 1 and 2 with two #3 bars, $\varepsilon_s/\varepsilon_c = 0.10$
Series 3 and 4 with one #3 bar, $\varepsilon_s/\varepsilon_c = 0.30$

Cover in all cases is $\frac{3}{4}$ in. Less coverage will reduce and greater coverage will increase these values. Pending more information, these values are suggested for heavy and moderate reinforcement respectively.

APPLICATIONS

Example 1

Calculate the probable midspan warping deflection due to shrinkage of a 6-in. slab with $d = 5$ in., $L = 10$ ft (either a simple span or between points of contraflexure) with medium shrinkage and #4 bars at 6 in.

Eq. (1) applies.

ε_c = 75 percent of total shrinkage = 0.75 (600) = 450 millionths
$\varepsilon_s/\varepsilon_c = 0.30$

$$k = \frac{\varepsilon_c}{d}(1 - \varepsilon_s/\varepsilon_c) = \frac{450}{5}(1 - 0.30) = 63 \text{ millionths radians per in.}$$

$$\Delta = \frac{kL^2}{8} = \frac{63\,(14{,}400)}{8\,(1{,}000{,}000)} = 0.1134 \text{ in.}$$

If steel is exposed on the soffit face,

$$d = 5.75 \text{ in.} \quad \varepsilon_s/\varepsilon_c = 0 \qquad k = \frac{450}{5.75}(1 - 0) = 78.2, \text{ and } \Delta = 0.141 \text{ in.}$$

Example 2

Calculate the probable warping deflection of a heavily reinforced cantilever slab with a span of 8 ft, high shrinkage concrete, $t = 4$ in., and $d = 3$ in.

Eq. (1) applies.

$\varepsilon_c = 0.75\,(700) = 525$ millionths
$\varepsilon_s/\varepsilon_c = 0.10$
For an 8 ft cantilever, $L = 16$ ft

$$k = \frac{525}{3}(1 - 0.1) = 157.5 \text{ millionths radians per in.}$$

$$\Delta = \frac{157.5\,(36{,}864)}{8\,(1{,}000{,}000)} = 0.726 \text{ in.}$$

The examples are actual cases and agree with measurements of the subject structures. Numerous similar examples can be cited.

CONCLUSION

Unexplained warping due to shrinkage is the source of implications that must be faced. While the load capacity is actually increased and the flexural properties not affected, the consequences of warping can be severe. Knowledge of the warping process, determination of the physical factors involved, and a reliable means of anticipating and evaluating warping deformation are essential in order to eliminate it or to restrict it to tolerable limits.

The information contained in this report is intended to improve reinforced concrete design practice particularly with regard to thin slabs and shallow beams. Obviously, warping can be eliminated by means of symmetrical sections. It can be substantially reduced by a relatively small amount of reinforcement near the concave face. Low-shrinkage concrete and a larger number of small bars rather than a smaller number of large bars can provide considerable reduction.

The subject of warping has been given little attention, and there is a corresponding lack of information. This study proves that warping is a relatively simple phenomenon when viewed apart from the complex abstractions that are associated with strength and elasticity. Continued research along this line, particularly with respect to interacting strains, can resolve present questions and clarify this important aspect of reinforced concrete practice.

ACKNOWLEDGMENT

The valuable assistance of Frank Abadi and Donald A. Sangesand, who participated in the research in partial fulfillment of the requirements for the degree of MS in CE, is gratefully acknowledged.

Support by the Engineering Experiment Station of the University of Washington implemented much of the program on which this report is based.

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Transverse Strength of Concrete Block Walls*

By F. W. COX† and J. L. ENNENGA‡

SYNOPSIS

Twelve concrete block wall panels were tested in pure horizontal flexure. The panels represented both 8-in. hollow load-bearing block and cavity construction with 4 in. thick wythes§ and 2-in. cavity. Joint reinforcement was used in half the panels. All results are expressed in terms of the maximum bending moment per foot of height which the wall withstood before breaking. The strength in horizontal span was found to be several times greater than the strength reported by other experimenters for vertical spans.**

INTRODUCTION

Concrete masonry is primarily a load-bearing material with relatively large compressive strength. The tensile strength is small and highly variable in character. Up to within the last 10 or 15 years, masonry was used exclusively as a load-bearing material, and walls possessed gravity stability because of the mass of the wall and the superimposed floor loads. Under such design conditions, flexural and tensile strengths were not a consideration.

The trend of the last 15 years, however, has been in the direction of walls with less and less lateral support and with little or no superimposed loads to make them gravity-stable. Warehouses and shops with walls 20 ft or more in height are common, but cannot be justified on the basis of gravity stability. Moreover, many modern wall designs for schools or offices have strip windows and large door openings which interrupt the beam action.

Building codes usually require only that transverse support be provided at either horizontal or vertical intervals not to exceed 18 times the nominal wall thickness. Building codes also require that structures be designed to resist certain wind pressures, but fail to give any other basis for the design of individual wall panels. The authors believe that spacing transverse supports at distances not to exceed 18 times the wall thickness is generally satisfactory for solid walls, but it cannot be applied indiscriminately to walls regardless of their shape or the size and arrangement of openings cut in the walls. Under such conditions, the engineer designing for wind pressures and other lateral forces needs some estimate of the flexural strength of masonry. The object of the tests herewith reported is to provide such information.

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§A wythe is each continuous vertical section of a wall one masonry unit in thickness and tied to its adjacent vertical section or sections by bonders, metal ties, or grout.

**_Bulletin_ No. 251; University of Illinois Engineering Experiment Station; _Report_ BMS-5, National Bureau of Standards.

Fig. 1 (left)—Construction of A panel showing Z-bars. Fig. 2 (right)—Construction of D panel showing joint reinforcement

All previous research available to the authors has been concerned with the flexural strength in the vertical direction. Such strength has been extremely low, with the panels always failing in the bond of the mortar to the block in a bed joint. Tests by the authors indicate that the strength in the horizontal direction is much greater than in the vertical direction. This is because the strength of the usual mortar mixes is sufficient to prevent failure by shear through the mortar. In other words, the shear strength of mortar in the bed joints prevents individual block from rotating out of position in the plane of the wall. Therefore, failure occurs by complete rupture through the block and vertical mortar joints (see Fig. 4, 5, and 7).

MATERIALS AND TEST EQUIPMENT

Twelve masonry panels were constructed. Three "A" panels were of 4 in.-2 in.-4 in. cavity* wall construction, using ¼-in. Z-bar ties for each 3 sq ft of wall area (Fig. 1). Three "B" panels were of similar 4-2-4 construction, except that extra heavy joint reinforcement was used in each bed joint to replace the Z-bar ties. Three "C" panels were constructed of 8 x 8 x 16-in. hollow load-bearing concrete block. Three "D" panels were constructed of 8 x 8 x 16-in. hollow load-bearing concrete block with standard joint reinforcement in each bed joint (Fig. 2). All panels were six block long (8 ft) and all A, B, and D panels were 3 ft 4 in. high (five courses). Panels C-1, C-2, and C-3 were three, five, and seven courses high, respectively, to determine whether the variation in strength was other than a straight-line variation.

The block were 8 x 8 x 16-in. hollow masonry units and 4 x 8 x 16-in. cored slabs taken from the manufacturer's stockpile. All block were load-bearing type made with sand-gravel aggregate. The joint reinforcement for the 8-in.

*Cavity wall is a wall built of masonry units so arranged as to provide an air space within the wall and in which the inner and outer wythe are tied together with metal ties.

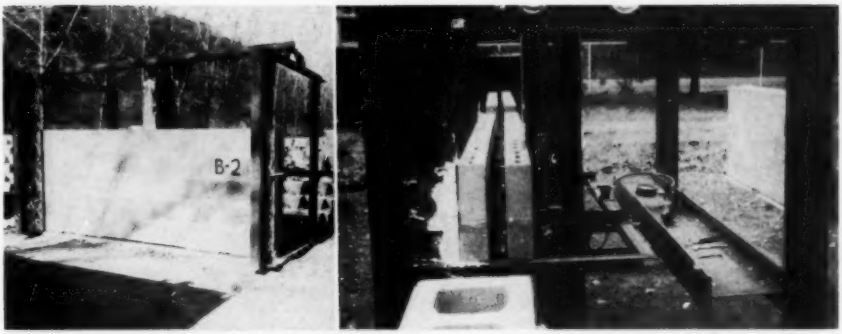

Fig. 3—(Left) End reaction frames of test jig. (Right) Wall panel in test jig, loading beam and hydraulic jack in center

walls had 9-gage longitudinal wires and 9-gage web members. Reinforcement for the cavity walls was made with 3/16-in. longitudinal wires and 9-gage web members, with a drip or crimp located at the center of each web member. All the walls were built in mid-September by a masonry contractor.

The specifications called for a mortar composed of one part portland cement, one part lime, and five parts sand by volume. The mason failed to follow specifications and, due to his speed of operation, the error was not caught until five of the 12 walls had been started. The mortar used was two parts portland cement, one part masonry cement, and three parts sand. This mix is preferred by some masons in this area for cold weather construction. To obtain comparable results between all panels, the authors permitted the masons to continue use of this mortar throughout. Rich mortars generally develop high compressive strength* but are often weak in bond. The authors believed that, even with weak bond, the strength developed in the bed joints would be sufficient to develop the wall in bending. This belief was verified when all the panels broke with vertical cracks extending from top to bottom of the wall with no sign of shear failure along bed joints.

All test panels were broken in a structural steel jig which produced conditions of a simple span with a concentrated load in the center. The load was applied with a hydraulic jack through a vertical loading beam (Fig. 3). The least reading of the jack was 200 lb. The loading beam was suspended from a roller chassis which was part of the jig. Likewise, the test panels were constructed to bear on rollers made of short sections of pipe which rolled on smooth troweled concrete slabs. This eliminated most of the bottom restraint which would otherwise exist. The success of the attempt to reduce restraint was evident when all the walls, at failure, developed a hairline crack extending from the top to the bottom of the wall. These cracks widened uniformly as the pressure was increased (Fig. 4 and 5). It was also noted that after the first crack appeared in unreinforced walls, the walls moved with little jacking pressure. Bottom restraint could not exceed this jacking pressure.

*Tests run by the Testing Laboratory, Missouri River Division, Corps of Engineers, indicated an average compressive strength of 5508 psi and a tensile strength of 515 psi at 28 days for the mortar.

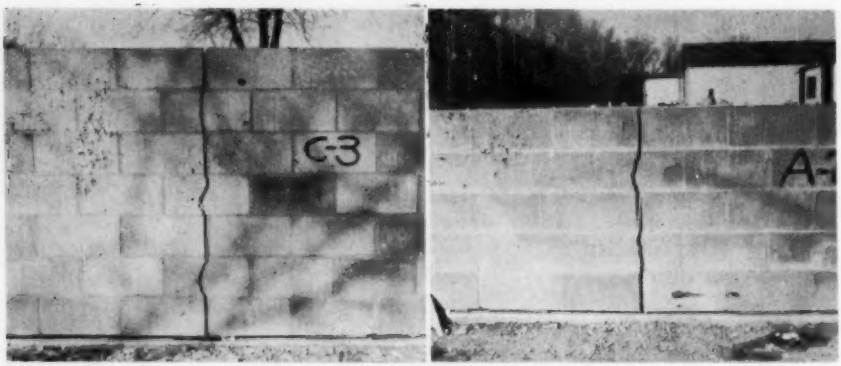

Fig. 4 (left)—Typical failure through block and header joints of 8-in. block wall without joint reinforcement. Fig. 5 (right)—Typical failure of 4-2-4 cavity wall without joint reinforcement

The relative freedom from restraint, both in the walls and in the loading beam, is an important and unique feature of these tests. These features enable the experimenter to measure the strength of each panel resulting from its resistance to horizontal bending alone. The lateral strength of a wall is the summation of the strength resulting from several sources as enumerated on p. 958. One major source of strength, however, for which the authors could find no previous source of test data is the ability of running bond masonry construction to span horizontally as a beam. Therefore, a value was desired as nearly free of restraint as possible. Moreover, regardless of the loading pattern, it is felt that any particular masonry wall must have one limiting maximum bending moment at which it will fail, except in the case of very short walls which might first fail in shear. Within limits, the loading pattern or span length does not affect the maximum bending moment at which a given steel or reinforced concrete beam will fail. Failure will result from any combination of loads and span which produces the maximum critical bending moment for the beam. It then seems reasonable to believe that the same will be true for concrete block walls when they function as a beam. Because an isolated value for horizontal beam strength alone was desired, the equipment used, which produced conditions of a concentrated load in the center of a simple span, is believed to be as satisfactory as any which might be devised. This is particularly true since none of the walls show evidence of failure by shear. The maximum bending moment per foot of height is quoted in the results because that unit is believed to be the most practical measure of wall strength.

Deflection of the walls was measured with three Ames dials (least reading 0.001 in.) mounted on an angle frame isolated from the loading mechanism. The gages were located at midheight of the wall and at quarter-points of the span. The first wall panel was broken 36 days after the walls were constructed. The remaining walls were broken 36 to 41 days after construction.

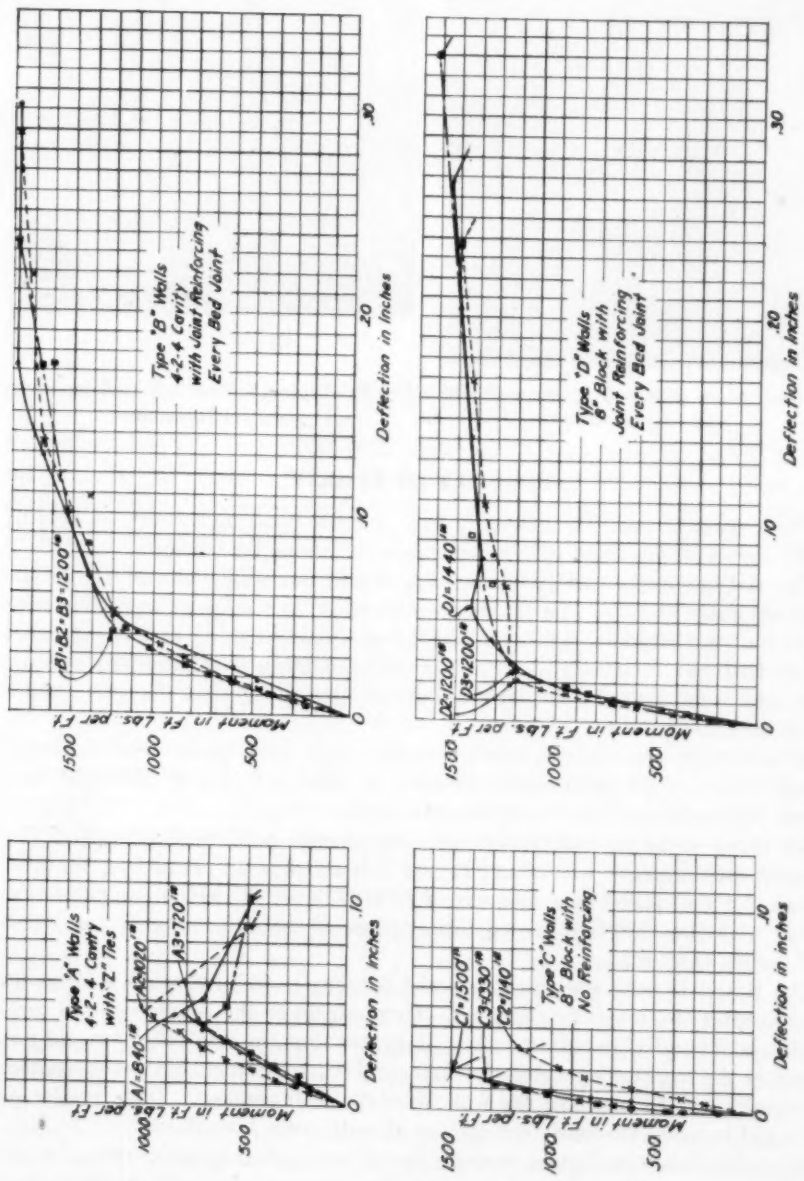

Fig. 6—Moment-deflection diagrams for block walls

Fig. 7—Failure of 4-2-4 cavity wall with joint reinforcement

Fig. 8—Typical failure in 8-in. D wall with joint reinforcement

SUMMARY OF RESULTS

The moment-deflection curves in Fig. 6 show good uniformity of results. These curves approached the typical parabolic shape for concrete.

The A-type walls (cavity walls with Z-bar ties only) proved to be the weakest walls tested. The failures occurred in two stages, beginning with rupture of the loaded wythes. Initial failure occurred at an average moment of 860 ft-lb per ft of height. The far wythe continued to carry an average moment of 480 ft-lb for a perceptible time, indicating that the two wythes act as separate beams.

In contrast, the B-type walls (cavity wall with joint reinforcement) failed under a load producing a moment of 1200 ft-lb per ft of height, and failure was evidenced by cracks in both wythes (Fig. 7).

No trend could be established with the several heights of C walls. The greatest unit strength was observed in the smallest wall. However, the five-course C-2 wall failed at a moment of 1140 ft-lb per ft, which compares favorably with the 1200 ft-lb per ft initial failure of the five-course D-2 and D-3 wall panels (Fig. 8).

The B and D walls continued to resist bending moment through a large deflection after the masonry ruptured. Comparison of the reinforced and non-reinforced D and C panels, however, indicates that the reinforcement did not influence the load under which the wall cracked, but that it did act to control the cracks and to preserve the wall after the wall cracked. Joint reinforcement did increase the basic strength of B walls over A walls.

Several panels were loaded through one or two partial cycles to demonstrate elastic recovery within certain limits. For instance, the load on Panel A-2 was raised to 480 ft-lb per ft of height, then reduced to zero followed by loading to failure. Similarly, B-1 was cycle loaded to 1200 ft-lb per ft; C-1

TABLE 1—LABORATORY STRENGTHS OF BLOCK

Sample	Size, in.	Span, in.	Ultimate load, lb	Ultimate moment ($PL/4$), ft-lb
1	8x8x16	13	2820	762
2	8x8x16	13	3910	1055
3	8x8x16	13	2690	729
4	8x8x16	13	2640	715
5	8x8x16	13	3630	982
6	8x8x16	13	3560	961
Average				867
1	4x8x16	14½	2130	642
2	4x8x16	14½	2490	750
3	4x8x16	14½	2500	755
4	4x8x16	14½	1820	570
5	4x8x16	14½	1920	578
6	4x8x16	14½	2000	602
Average				649

to 1000 ft-lb per ft; C-2 to 840 ft-lb per ft; C-3 to 429 ft-lb per ft; and D-3 to 800 ft-lb per ft. Each was unloaded and then loaded to failure. Maximum strength did not appear reduced by the one or two cycles of loading. All walls partially loaded recovered all but 0.002 to 0.004 in. of their initial deflection.

It would be desirable to establish a relation between the flexural strength of individual block and the strength of the wall as a whole. It is the authors' belief that the principal strength in a wall comes from the strength of the individual block which must be ruptured before the wall fails. The theory was therefore considered that the total strength of a wall must bear a close relationship to the product of the rupture strength of individual block, multiplied by the total number of block in the plane of rupture. To explore this theory, six 4 x 8 x 16-in. and six 8 x 8 x 16-in. block were tested in flexure as a simple beam lying on one side, with concentrated loads in the center. Large strength variations were noted in individual block (Table 1). The small strength difference between the 4-in. and 8-in. block indicated some error or inconsistency in the method of testing.

The strength of the 4-in. block seems to approximate that of the type A walls from which they were built. As seen in Fig. 5 and 7, failure occurred by rupture through two blocks in each wythe, or a total of four blocks. Table 1 shows the average strength of block to equal 650 ft-lb per block or 2600 ft-lb total for the four block. The average total strength of wall from Table 2 was 860 ft-lb per ft of height. The total height was 3 ft 4 in. and therefore the total actual strength was 2860 ft-lb. The difference between actual strength of 2860 ft-lb and predicted strength of 2600 ft-lb can readily be attributed to bond in the header joints and the restraint at the bottom of the wall panels. All evidence accumulated for this report indicates that the 4-in. block functioned in this manner, both when tested separately and when constructed into a block wall.

TABLE 2—SUMMARY OF RECOMMENDATIONS

Wall type	Average moment at rupture, ft-lb per ft of height	$L = \sqrt{\frac{8M}{W}}$	L in ft for a safety factor of 2	Remarks
A	860	18′ 6″	13	
B	1200	22′ 0″	—	18′ 0″ design span
C	1140	21′ 4″	15	
D	1280	22′ 7″	—	18′ 0″ design span

It was evident, when the 8-in. block failed at only 867 ft-lb of bending moment, that something was defective in the manner of testing or in the type of failure which had occurred. Based on section modulus, it should logically follow that an 8-in. block would have approximately four times the flexural strength of a 4-in. block. When placed in a concrete block wall, the one face shell of the block must act in tension and the opposite face shell must act in compression, thus developing the force couple which resists the bending moment of applied load. The low breaking strength of the 8-in. block, when compared with the strength of the 4-in. block, indicates to the authors that the 8-in. block, when tested individually, did not fail in flexure as a single beam. It appears that the two face shells failed individually, each one acting as a separate shallow beam. The authors do not pretend that experimental data is sufficient to prove the point in either direction. There is some evidence that the 4-in. block does give an indication of a total wall strength, whereas the 8-in. block plainly does not.

DISCUSSION

The strength of a masonry wall may stem from several sources:

(a) Tendency of the wall to cantilever from its base.

(b) The direct floor and roof loads cause compression in the wall which must be overcome before bending in the vertical direction can cause any tension in the masonry.

(c) Flexural strength in the vertical direction (always small).

(d) Horizontal arching action of walls in which the ends of each panel are restrained between rigid supports such as columns.

(e) Flexural strength in the horizontal direction because of the interlocking effect of successive courses of block, which is the subject of this report.

The average bending moment per foot of height is given in Table 2. The choice of a safety factor to accompany these bending moments must depend not only on the strength contributed by each of the sources listed above, but also upon the possibility of random shrinkage cracks. Random shrinkage cracks may completely destroy the horizontal flexural strength of the wall. The probability of such cracks increases as the control over the quality of the block is relaxed. The use of properly designed shrinkage control joints located at strategic intervals, in combination with well-cured, high-grade block,

eliminates most of the random cracks. These control joints must be a complete separation through the full thickness and height of the wall. They should be located at points of lateral support (such as columns or transverse walls) or midway between such supports so as to develop simple spans or balanced cantilever spans. It is also desirable to consider the distance which each type wall would span before failure under wind pressure. The wind load recommended by a committee of the American Society of Civil Engineers* and adopted by the Corps of Engineers† is 20 psf for the first 300 ft above ground level. Maximum spans were calculated from the test data for each type of wall, using this wind pressure (see Table 2).

Referring to new walls free of all apparent defects, the tests indicate that an 8-in. solid wall will span 22 ft and a 4-2-4 cavity wall will span 18 ft 6 in. before failing under a 20 psf wind pressure. Applying a safety factor of two, these spans are reduced to 15 and 13 ft, respectively. This compares favorably with the present allowable span of 12 ft permitted by the "American Standard Building Code Requirements for Masonry."‡ Thus, the building code would seem to make some allowance for small window openings in wall panels, and minor defects.

The authors would be less conservative in the handling of walls with joint reinforcement as typified by B and D walls. The effect of joint reinforcement, however, fell below expectations. Calculations will show that the stress in joint reinforcement is negligible at the stress level where masonry ruptures. It was observed that the basic strength of 8-in. walls with joint reinforcement was increased little over those without reinforcement, even though reinforcement was used in every bed joint; but the reinforcement aids in reducing shrinkage cracks and therefore assures the designer of a certain minimum flexural strength. If control joints were provided at each transverse support§ and joint reinforcement was continuous between transverse supports in every bed joint, the authors would allow 8-in. walls to span 18 ft between supports.

The strength of 4-2-4 cavity walls was increased 33 percent by the addition of joint reinforcement. The joint reinforcement unites the two wythes and tends to cause them to act as a single beam. The authors would therefore permit 4-2-4 cavity walls with joint reinforcement in every bed joint to span the same distance between supports as 8-in. stretcher block walls.

If windows and doors occurred in any particular panel, the same amount of total reinforcement would be required as in a panel without openings, except all the reinforcement which normally would have been applied to the zone in the height of the windows or doors must be concentrated immediately above and below such openings. This can be accomplished with reinforced con-

*"Wind Bracing in Steel Buildings," Final report of Subcommittee No. 31, ASCE Committee on Steel of the Structural Division, *Transactions*, ASCE, V. 105, 1940, p. 1713.

†*Engineering Manual for Military Construction*, Department of the Army, Corps of Engineers, Office of the Chief of Engineers, 1940, Part IV, Chapter 1, p. 2.

‡*Miscellaneous Publication* No. 211, National Bureau of Standards, U. S. Department of Commerce, July 1953.

§The control joints used by the Corps of Engineers incorporate a keyed connection between adjacent panels, and the joint will transfer shear forces but not moment. Therefore, a control joint at the point of support creates simple span conditions between the supports.

crete block beams formed by placing concrete and reinforcing steel in a shell formed from standard bond or lintel beam units. Detailed rules are not practical, but it is evident that where large openings occur in a wall, the remaining sections of wall must be strengthened to compensate for the loss of strength across the opening.

In choosing design strength factors, the authors recommend the practice of selecting the principal source of strength and assigning a design value for that one source of strength. Then, in the design of masonry walls, they disregard all other sources of strength and treat them as additions to the factor of safety. For example, the "American Standard Building Code Requirements for Masonry"* permit an 8-in. block wall to span only 12 ft in either the vertical or the horizontal direction. The authors would permit the walls to span to greater heights if vertical supports were placed at intervals not exceeding 18 ft, as recommended in Table 2 for type D walls and if joint reinforcement was placed in every bed joint. Then the tendency of the wall to cantilever from its base, the flexural strength in the vertical direction, and other sources of strength would be used as a factor of safety.

CONCLUSIONS

The number of walls tested was small, but correspondence of results was close for each type of wall tested. The bending moment per foot of height expressed in foot-pounds is therefore considered practical information for walls working in simple horizontal span conditions.

The results are shown in Table 2. The A and B walls were of 4 in.-2 in.-4 in. cavity construction. The C and D walls were of 8-in. stretcher block construction; the D walls contained joint reinforcement. The indicated spans are based on the moment caused in a simple span with 20 psf wind loading. A factor of safety of two would result in spacing transverse supports 13 to 15 ft apart in nonreinforced walls. However, the authors recommend 12-ft spacing in compliance with the "American Standard Building Code Requirements for Masonry." A span of 18 ft between supports is considered reasonable for 8-in. walls with joint reinforcement in every bed joint, and for 4-2-4 cavity walls with joint reinforcement in every bed joint.

The information in the report relates to only one grade of mortar and only to 8-in. block walls and to cavity walls composed of two 4-in. wythes separated by a 2-in. cavity. Such walls are treated by the "American Standard Building Code Requirements for Masonry" as having equal strength to an 8-in. wall. The experimental data is limited but should be valuable to structural designers if good judgment is used, based on the strength of materials and the size and shape of the walls in question.

**Miscellaneous Publication* No. 211, National Bureau of Standards, U. S. Department of Commerce, July 1953.

Discussion of this paper should reach ACI headquarters in triplicate by Aug. 1, 1958, for publication in the Part 2, December 1958 JOURNAL.

Reinforcement of Press Foundations by Post-Tensioning*

By FRITZ KRAMRISCH†

SYNOPSIS

Existing foundations were post-tensioned to accommodate new presses almost twice as heavy as those for which foundations were originally designed. Method is described and load analysis given.

INTRODUCTION

Five 2500-ton presses and two 1950-ton presses were to be placed on foundations that had been designed and constructed to receive presses of much smaller weight. The existing press foundations consisted in general of hollow concrete boxes about 14 x 22 ft in plan, 18 ft deep, with 12 in. thick walls, 24 in. thick bottom slabs, and solid piers for the support of the presses.

Although the presses to be placed on these foundations were almost twice the weight for which the foundations were designed, the size and location of the piers and the required clearances were sufficient to accommodate the heavier presses. The use of these foundations, therefore, depended solely on the possibility of increasing the structural strength of the base slab.

The original design of the bottom slab was based on an equally distributed soil pressure. Because the reinforcement used could not provide for an increase of more than 50 percent in the total load, and because concentration of the bearing pressures under the piers alone would produce excessive values, a method of post-tensioning was chosen to increase the structural strength of the base slab. The tensioning was to be applied only in one (longitudinal) direction between the main piers supporting the presses.

ANALYSIS

Fig. 1(a) is a longitudinal section through the base slab with all forces acting upon it. Fig. 1(b) shows schematically the moment distribution in the base slab due to the loads from piers and press and the equally distributed soil reaction.

A = cross sectional area of base slab
m = distance of centroid from the bottom
S_T = section modulus of base slab at top
S_B = section modulus of base slab at bottom
F_1, F_2 = friction at bottom of base slab
P = prestressing force
e = eccentricity of P

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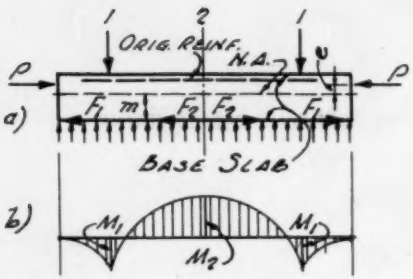

Fig. 1—Forces and moments in the base slab

A, m, S_T, and S_B have been determined for the transformed section under consideration of the existing mild steel reinforcement. F_1 and F_2 had to be assumed, but an error in their approximation is of only little importance because F is small in comparison to P.

P and e were then figured from the assumption that the longitudinal tensile stress f_{1B} in the bottom fiber at 1, and f_{2T} in the top fiber at 2 shall be reduced to zero.

$$f_{1B} = -\frac{(P - F_1)}{A} + \frac{(M_1 + F_1 m + Pe)}{S_B} = 0$$

$$f_{2T} = -\frac{(P - F_2)}{A} + \frac{(M_2 - F_2 m - Pe)}{S_T} = 0$$

The corresponding compression stresses f_{1T} and f_{2B} were far below the maximum allowable values.

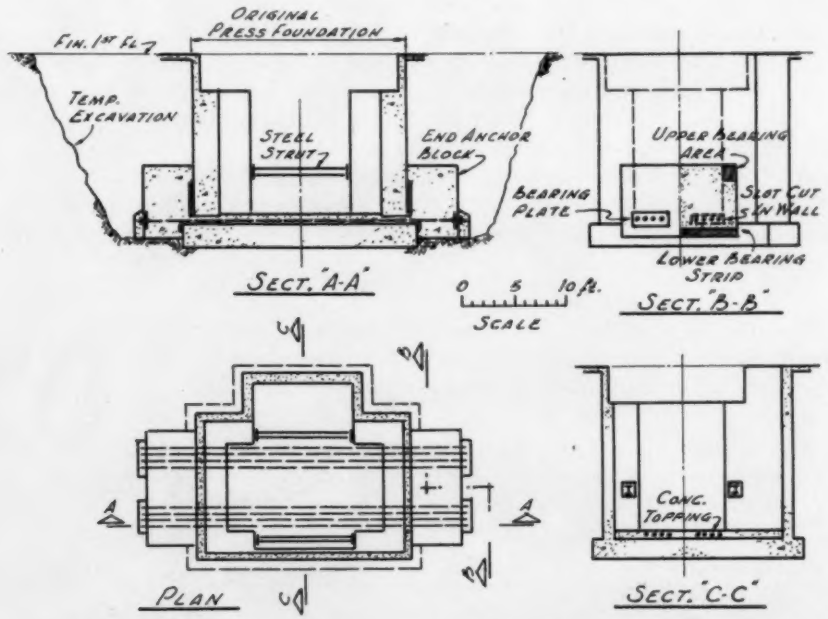

Fig. 2—Plan and sections of press foundation with added prestressing steel

POST-TENSIONING PROCEDURE

Excavations were made at both ends of each press foundation and carried down to the bottom elevation of the base slab. Afterward, slots were cut into each end wall close to the top of the base slab and two anchor blocks were cast—one at each end of the foundation box with inserted pipe sleeves extending from the slots to the bearing plates at the exterior ends of the anchor blocks. Stressteel bars were then placed through the sleeves, extending across the floor of the base slab to the exterior bearing plates. The purpose of the end anchor blocks was to provide anchorage for the tensioning units, and to transfer the required pre-compression into the foundation slab. Plan and sections are shown in Fig. 2, a schematic drawing of the arrangement.

The end anchor blocks were designed to press against the foundation slab over a narrow rectangular area at the calculated elevation and to be counterbalanced at their upper end. To transfer most of the tensioning force into the foundation slab, the end anchor blocks (Fig. 3) were made relatively high to produce a great ratio of a/b. In order to protect the light reinforced slabs of the end walls, continuous bearing along the upper end of the anchor block had to be avoided and the bearing was concentrated at each upper corner in a square area pressing directly against the mass of the pier. Provision of a layer of soft material prevented bearing at intermediate areas where the contact between the anchor block and the original foundation box was to be avoided. Steel struts were shimmed in tight in the line of the upper bearing areas to transfer the compression across the openings.

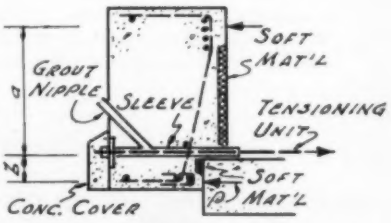

Fig. 3—Detail of end anchor block

After the tensioning units were stressed, a protective concrete topping was placed over the floor of the base slab and the sleeves were pressure grouted. Concrete protection was placed, also, over the exposed nuts and bearing plates and the excavation backfilled with sand.

The presses in a major factory in the Detroit area have been in successful operation now for over 2½ years on the strengthened foundations. Albert Kahn Associated Architects and Engineers, Inc., Detroit, designed the press foundations and the reinforcement; Darin and Armstrong, Inc., Detroit, were the general contractors. Truscon Steel Division of Republic Steel Corp. furnished the Stressteel bars.

Discussion of this paper should reach ACI headquarters in triplicate by Aug. 1, 1958, for publication in the Part 2, December 1958 JOURNAL.

Title No. 54-56

High-Density Concrete for Shielding Atomic Energy Plants*

By HAROLD S. DAVIS†

SYNOPSIS

Concrete properties required for gamma ray absorption and neutron attenuation are outlined. "Fixed" water content and its role in slowing down fast neutrons are explained. Grouts and mortars, heavy aggregate properties, and use of boron are also covered. Placement methods evaluated include: conventional, pumping, prepacking, and puddling. Formwork problems including dimensional tolerances are outlined, and shields made of high-density concrete block are described. Tabulated data cover costs of materials as well as finished shielding structures.

INTRODUCTION

Concrete is composed of pieces of inert aggregate held together by a hardened cement paste. Conventional concrete, made with sand and gravel, or crushed aggregate, portland cement, and water, weighs about 150 lb per cu ft and costs around $50 to $125 per cu yd, installed. High-density concretes, which have special compositions for improving shielding effectiveness, may cost from $150 to $1000 per cu yd, including forms. Proper use of these special concretes may result in reducing the over-all cost of a project, or in improved efficiency of operation.

Properly designed and installed concrete can be used with confidence for constructing radiation shields. However, good concrete is not obtained by chance, but requires sound aggregate, adequate control, and proper methods of construction. In general, the best of standard practice is adequate, but *definitely required*, to produce shielding concrete with the specified composition and uniformity. Variations from standard practices, such as adding boron to the mix, should be approached cautiously. When working with high-density concrete, the concrete laboratory is an essential part of the design and construction operation. It can be used effectively for confirming the adequacy of raw materials and procedures, and for indicating methods of cost reduction.

*Presented at the ACI 10th regional meeting, Seattle, Wash., Nov. 5, 1957; also presented at Conference on Shielding of High-Energy Accelerators, New York, Apr. 11-13, 1957. Proceedings of the conference have been issued as TID-7545 by the Technical Information Service Extension of the Atomic Energy Commission, and are available at $5.50 from the Office of Technical Services, Department of Commerce, Washington 25, D. C. Title No. 54-56 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 11, May 1958, *Proceedings* V. 54. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Aug. 1, 1958. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

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COMPOSITION AND PROPERTIES OF HIGH-DENSITY CONCRETES

Special concretes for constructing shielding structures are distinguished from conventional concrete by having higher density and/or special compositions to improve the attenuation of neutrons. In the design of shields for absorbing gamma rays, high-density and minimum cost are of prime importance. In the design of concrete shields to attenuate neutrons, sufficient material of light atomic weight, such as hydrogen, should also be included in the composition. The ability of a shield to absorb gamma energies of 1 to 3 Mev (and also fast neutrons to a limited extent) is almost proportional to the density of concrete. This fact may be used as a rough estimate of the required density, thickness, and cost of a particular shield when several types of concrete are being considered. Data on attenuation properties of several types of concrete are given in Table 1. In using these data, consideration should be given to the radiation sources and experimental conditions used in determining the attenuation factors. The

TABLE 1—RELAXATION LENGTHS*

Material	Density, g per cu cm	Relaxation length, cm	
		Fast neutrons	Gamma rays
Water	1.0	~10	30
Graphite	1.62	~ 9	19
Beryllium	1.85	~ 9	18
Beryllium oxide	2.3	~ 9	14
Concrete (144 lb per cu ft)	2.3	11†	15
Aluminum	2.7	~10	13
Barytes concrete (218 lb per cu ft)	3.5	8.0‡	10
Iron concrete (268 lb per cu ft)	4.3	6.3§	8
Iron	7.8	~ 6	3.7
Lead	11.3	~ 9	2.5

*Approximate values reprinted from *Principles of Nuclear Reactor Engineering*, by S. Glasstone, 1955, p. 609.

†More recent value 13 cm.
‡More recent value 10.6 cm.
§More recent value 8.2 cm.

TABLE 2—MIX DATA FOR 17 HIGH-DENSITY CONCRETES

Concrete	Placement*	Weight, lb per cu ft	Material cost per cu ft	Aggregate†		Portland cement,‡ lb per cu ft	Mix water, lb per cu ft	Water content,§ lb per cu ft	
				Coarse, lb per cu ft	Fine, lb per cu ft			Min.	Max.
A	D	410	$25.40	324 (I)	63 (#110S)	16.8	5.9	2.5	5.9
B	P	346	18.40	270 (I)	44 (M)	20.6	11.3	3.5	11.8
C	C	300	15.40	161 (F)	107 (F)	M.O.‡	—	9.5	15.0
D	C	300	12.05	171 (F)	92 (F)	24.1	12.7	3.6	12.7
E	P	300	11.90	67(M) +160(I)	42 (M)	19.8	10.9	4.1	12.0
F	P	263	11.20	60(L) +140(I)	28 (L)	22.2	12.2	13.0	21.9
G	C	262	11.30	100(#1320S) +49 (#330-S) +65(L)		34.0	13.5	12.3	20.9
H	C	262	7.75	70(F) +50 (B)	70(F) +35(B)	23.7	12.8	3.6	12.8
I	P	262	5.75	67(P) +122(M)	42 (M)	19.8	10.9	4.7	12.6
J	P	244	2.55	180 (M)	37 (M)	17.5	9.7	4.8	11.9
K	C	232	2.20	110 (M)	86 (M)	24.3	11.5	5.7	13.5
L	P	227	3.30	168 (B)	29 (B)	19.3	10.5	2.9	10.5
M	C	222	3.05	105 (B)	86 (B)	19.3	11.6	2.9	11.6
N	C	219	2.10	100 (HI)	82 (HI)	24.9	12.0	9.2	17.5
O	P	215	2.75	28(L) +122(M)	29 (L)	22.7	12.5	10.9	20.0
P	C	185	3.10	76 (L)	62 (L)	31.3	15.4	17.1	27.8
Q	C	154	0.50	61 (G)	50 (S)	31.3	11.5	4.7	11.5

*Placement methods: puddled (D), prepacked (P), conventional (C).
†Heavy aggregates: steel, iron punchings, or sheared bars (I); steel shot (S); ferrophosphorus (F); magnetite (M); limonite (L); hydrous iron ore (HI); barite (B); ordinary gravel (G); and sand (S). (See Table 8 for physical properties of heavy aggregates.) Utah limonite is used in design of Mixes F, G, and O; Michigan limonite in Mix P.
‡Portland cement used in all mixes except C in which magnesium-oxychloride cement is used. (12.2 lb per cu ft of MgO powder and 19.8 lb per cu ft of MgCl solution.)
§Maximum water content is water weight when concrete is wet. Minimum water content is amount left after drying to constant weight at 85 C. The difference between the maximum water content and the amount of mix water is the water of crystallization held by the aggregate. The difference between the minimum water content and the water of crystallization in the aggregate is the water retained by the hardened cement paste.
NOTE:
Intrusion mortars: steel shot grout (S-270) used in Concrete A. Magnetite grout (M-170) used in Concretes B, E, I, and J. Limonite grout (L-146) used in Concrete F and O. Barite grout (B-155) used in Concrete L. (See Table 7 for grout data.)
Admixtures: mandatory to use Intrusion Aid or acceptable substitute in prepacked concretes placed by Intrusion-Prepakt Co. Plastiment may be used to good advantage in conventional concretes.

exact water content of the concrete used in neutron attenuation tests is difficult to determine, and may not be known accurately.

The optimum density and composition for a concrete shield are influenced by: (1) type and intensity of radiation, (2) limitations on shield dimensions or space restriction, (3) complexity of placing conditions and shape of forms, (4) number and proximity of through-tubes and embedded items, (5) proposed construction methods, (6) temperatures and operating conditions, (7) external and dead loads, (8) available aggregates, and (9) economy. In general, proportioning mixes of high-density concrete does not differ radically from conventional concrete; that is, the mix which produces the given quality of the hardened concrete at the least cost is usually the most desirable mix. However, an engineer must adjust his thinking to allow for increased density, the need of uniformity, and the fact that heavy aggregates may cost several times as much as the cement.

Mix data for several types of high-density concrete are given in Table 2.[1] However, prior to specifying proportions or using these mixes in actual shield construction, they should be checked and modified, if necessary, by experienced technicians. Since local job conditions and aggregates will vary sufficiently from job to job, laboratory trial mixes should be made prior to actual construction. Physical properties of high-density concrete are summarized in Table 3.

The concrete selected for a particular job should be economical and placeable, and should have desired uniformity, strength, and durability. It may not be necessary for shielding concrete to possess great resistance to wear or washing action; however, the concrete should resist the deteriorating effects of nuclear radiations and temperature cycling. Experience indicates that the radiation

TABLE 3—PHYSICAL PROPERTIES OF HIGH-DENSITY CONCRETE

Aggregate	Grand Coulee	Hanford	Limonite	Limonite-magnetite	Barite	Barite	Magnetite	Iron-limonite
Placement	Conventional	Conventional	Conventional	Prepacked	Conventional	Prepacked	Conventional	Prepacked
Composition type*	—	Q	P	O	M	L	K[1]	F
Bags cement per cu yd	5.0	9.0	9.0	6.55	5.1	5.0	7.0	6.55
Density (lb per cu ft)	153	154	185	210	226	230	224	273
Source of data,† Reference No.	4	5	5	6	4	4	6	6
Compressive strength,‡ psi	4960	8870	5865	4640	6130	3340	5730	3180
Modulus of rupture,‡ psi	630	1005	700	516	445	630	684	406
Modulus of elasticity, $E \times 10^{-6}$‡	4.6	5.9	4.4	5.0	4.3	3.7	4.3	5.4
Ultimate bond strength,‡ psi	—	900	970	1396	—	—	1423	1266
Shrinkage, percent	0.041	0.012	0.021	0.023	0.029	0.029	0.018	0.013
Coefficient of expansion, in. per in. per deg F	5.5			5.6	10	10	5.7	5.9
Specific heat, Btu per lb per deg F	0.23			0.20	0.157	0.146	0.21	0.18
Conductivity, Btu per ft per hr per deg F	1.50			1.20	0.884	0.867	1.68	2.75
Diffusivity, sq ft per hr	0.042			0.030	0.025	0.026	0.034	0.056
Remarks	Ordinary concrete	Ordinary concrete	Michigan limonite	SG-4.4 for magnetite	Nevada barite	Nevada barite	Michigan limonite sand	Michigan limonite

*Compositions for these test specimens are similar to those given in Table 2 except for type K[1] concrete which contains 46 lb of limonite sand and 0.6 lb of fixed water per cu ft.

†The work described in Reference 4 was performed for the Atomic Energy Commission by the Bureau of Reclamation. The work described in Reference 5 and 6 was performed for the Atomic Energy Commission and the General Electric Company by the University of Washington and the Corps of Engineers, U. S. Army, respectively.

‡These strength values are for moist cured specimens, 28 days old.

TABLE 4—NEUTRON ATTENUATION DATA FOR CONVENTIONAL CONCRETE

Computed values based upon effective neutron removal cross sections

Unit weight, lb per cu ft	Percent water	W/C ratio	μ, cm^{-1}	Relaxation length, cm
153	10	0.742	0.0949	10.5
150	8	0.582*	0.0896	11.2
147†	6	0.427	0.0845§	11.8
143	4	0.278	0.0796	12.5
141‡	2	0.136	0.0749	13.3
139	1	0.067	0.0726	13.7
138	~0	~0.0	0.0704	14.2

*Initial W/C ratio, 0.60 with 6 bags of portland cement per cu yd.
†Corresponds to well cured, moist concrete at about 20 C.
‡Corresponds to concrete dried at about 150° C.
§ μ = 0.0884 cm^{-1} reported in ORNL-2193, with a water content of 7.48 percent. See this document for typical composition.

TABLE 5—ATTENUATION DATA FOR HEATED CONCRETE SLABS

Preliminary Hanford data submitted Oct. 1, 1956

Temperature	Attenuation of fast neutrons					
	Iron-limonite concrete			Magnetite-limonite concrete		
	Density, g per cu cm	μ, cm^{-1}	λ, cm	Density, g per cu cm	μ, cm^{-1}	λ, cm
Initial	4.27	0.128	7.8	3.40	0.111	9.0
100 C	4.14	0.119	8.4	3.33	0.098	10.2
175 C	4.04	0.103	9.7	3.27	0.087	11.5
320 C	4.03	0.099	10.1	3.26	0.091	11.0

stability of concrete made with sound materials is adequate for most applications. Experiments[2] demonstrate that concrete will have a life of at least 10 years in a flux of 10^{11} neutrons per sq cm. In some applications, high temperature, and not radiation damage, is critical. Thus, maximum temperatures in the concrete, as well as temperature differentials across the shield, must be kept within reasonable limits. A study by J. A. Lane[3] indicates that the maximum tolerable heat flux on the inside of a concrete shield is about 100 Btu per hr per sq ft, or 2×10^{11} Mev per sq cm per sec. This gives about a 50 F rise in the shield.[3] Thermal considerations in the design of concrete shields are discussed in more detail by the author in Reference 11.

WATER CONTENT

One of the objectives in designing a neutron shield is to provide sufficient water and other light elements so as not to impair its effectiveness for slowing down fast neutrons. The optimum water content is unknown at present; however, it appears that if the water content of concrete is below 4.0 percent by weight, neutron shield efficiency is impaired (see Tables 4 and 5). The minimum water content is a complex function of the composition of the entire shield, and the presence of other light elements, such as oxygen, assists in moderating neutrons.

Hydrogeneous materials for attenuating neutrons can be incorporated into concrete, but only by sacrificing density. Also, such concretes may be weaker and more costly than those not containing high amounts of hydrogenous materials. Water, an essential component of concrete, is present in several states. All the water added initially to a concrete mix does not remain indefinitely; part is fixed or chemically bound in the hardened cement paste, and part is evaporable. Actually, it may take 10 or 20 years for all of the evaporable water to diffuse through a thick shielding wall at low temperature, and considerably less time if exposed to high temperatures. The amount of fixed water held by chemical bond is a function of several factors, the main ones being: (1) initial water-cement ratio used in the mix; (2) curing history prior to start-up of the reactor; (3) surface conditions of the shielding wall; and (4) the duration and temperature of exposure since reactor's start-up. With careful curing, the fixed water content of portland cement concrete can be increased 50 to 100 percent in some cases. Heating tests performed on mortar samples have shown that fixed water content is also a function of temperature; the higher the temperature the lower the fixed water content (Table 6).

TABLE 6—EFFECT OF HEATING ON WATER RETAINED BY PORTLAND CEMENT PASTE*

Temperature, C	Water retained, percent by weight of portland cement		
	1 day	28 day	6 months
50	9.6	19.0	22.3
100	6.2	14.0	17.8
200	5.0	10.4	12.2
300	4.0	8.3	10.1
400	2.9	6.1	8.0
500	1.5	2.0	3.0
600	0.5	1.0	2.1

*Data obtained by Wilson and Martin, 1935. See p. 119 of Reference 8. Initial water-cement ratio of specimens 0.60.

The fixed water content of concrete can be increased by using: (1) a high content of portland cement in the mix, for example 9 to 11 bags per cu yd; (2) aggregate such as limonite which contains crystallized water; (3) careful curing methods; or (4) special cements such as magnesium oxychloride cement.[9] The amount of water of crystallization held by hydrous ore is also a function of temperature as shown in Fig. 1. From this chart, it is seen that limonite and goethite are reliable sources of hydrogen as long as the temperature does not exceed about 200 C. The amount of "free" or evaporable water contained in a concrete shield can be increased by using porous aggregate such as haydite, keeping the concrete cool, and sealing the surfaces of the concrete with impermeable membranes, or replenishing the "free" water.

GROUTS AND MORTARS

Various grouts and mortars have been developed and used in the construction of reactor shields. Data on several types of grout are presented in Table 7. The grouts contain about 1.5 percent by weight of an intrusion aid to improve pumpability and control shrinkage. It is not recommended that metallic grout be pumped into the large masses of aggregate. (Grout S-250 is more easily pumped than grout S-270.) The grouts listed can be used in the prepacked process for constructing large shields, and for filling around sleeves,

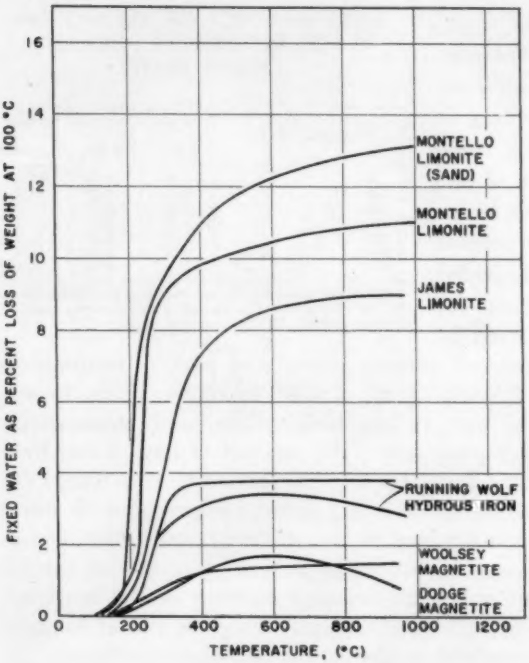

Fig. 1—Average values of fixed water content for heavy aggregates

or for constructing shielding plugs. The mortars can be used in the last two cases. A mortar developed at Argonne National Laboratory consists of 1 lb ferroboron, 2 lb sand, and 1 lb mortar cement, with a water-cement ratio of about 0.44. This mortar is suitable for spray or trowel application. In general, mortar is a concrete made with fine aggregate only, and is stiffer than grout.

AGGREGATE SELECTION

Presented in Tables 8 and 9 are data on heavy aggregates. Of around 60 minerals with specific gravity greater than 3.5, about ten are commercially available. Of these, barite, magnetite, ilmenite, limonite, and goethite are most suitable. Others such as hematite, taconite, arsenophrite, chromite, psilomelane, and galena are of interest but have not been widely used. Ferrophosphorus, a by-product obtained during the manufacture of phosphorus, has been also used successfully. Ferrosilicon may be of interest. However, when considering the use of heavy slags and smelter by-products, it is advisable to determine in advance whether or not they produce good concrete. Some of these by-products appear sound upon visual inspection, but when used in concrete they soften or disintegrate. Also available are steel and iron aggregates in the form of shot, punching, scrap, sheared bars, and special shapes.

In determining which aggregate to use, consideration should be given to availability of aggregates locally, their physical properties, and cost. Initial

procurement costs, freight charges, and additional processing costs at the site should be evaluated for each aggregate being considered. In general, heavy aggregates should be clean, strong, inert, and relatively free of deleterious materials which might impair the strength of concrete. Ordinarily, it is desirable to utilize the heaviest aggregate that is locally available; otherwise transportation costs may be excessively high. Since an excellent source of barite (barytes) is located within 20 miles of Oak Ridge National Laboratory, this heavy aggregate has been used extensively in shielding structures there. On the other hand, magnetite of greater density can usually be obtained in the western part of the United States at less cost than can barite. Limonite ore suitable for processing as coarse aggregate is not readily available in most sections of the United States.

Data in Table 2 indicate that concrete weighing from 200 to about 245 lb per cu ft can be obtained using natural heavy aggregates. Therefore, expensive

TABLE 7—INGREDIENTS OF HEAVY GROUT MIXTURES

Type	Grout sand	Sand-cement ratio		Water-cement ratio by weight	Unit weight wet, lb per cu ft
		By weight	By volume		
S-270	#110 steel shot	3.75	1.58	0.35	270
S-250	#110 steel shot	3.30	1.39	0.35	250
M-170	Magnetite	2.13	1.04	0.55	170
B-155	Barite	1.49	1.10	0.54	155
L-146	Limonite	1.28	1.09	0.55	146

NOTE:
Based upon weight of portland cement, 1.5 percent intrusion aid used in each grout.
Each of the above grouts is pumpable; however, it is not recommended to pump the metallic grouts into large masses of compacted aggregates. Grout S-250 may be pumped more easily than Grout S-270 designed for use in the "puddling" method.
Grout sands processed from heavy ores should have a fineness modulus of 1.0 to 1.5; the heavier the sand the finer it must be to stay in suspension. Sands having 20 to 40 percent finer than the No. 200 screen may be used if tensile strength under elevated temperatures is not important.

TABLE 8—PHYSICAL PROPERTIES AND COST DATA FOR HEAVY AGGREGATES*

Heavy aggregate	Source	Primary identification	Specific gravity, saturated surface-dry		Percent by weight		Dollars per short ton	
			Coarse pieces	Fine sand	Iron	Fixed water	FOB source	Processed at job‡
Limonite	Michigan	$2Fe_2O_3 \cdot 3H_2O$	3.75	3.80	58	9	7	40
Goethite	Utah	$Fe_2O_3 \cdot H_2O$	3.45	3.70	55	11	20	40
Magnetite	Nevada	Fe_3O_4, etc.	4.62	4.68	64	1	9	20
Magnetite	Montana	Hydrous iron†	4.30	4.34	60	2–5	7	18
Barite	Tennessee	92 percent $BaSO_4$	4.20	4.24	1–10	0	18	22–30
Barite	Nevada	90 percent $BaSO_4$	4.28	4.31	1	0	19	30–40
Ferrophosphorus	Tennessee, Missouri, and Montana	Fe_2P, Fe_2P, FeP	6.30	6.28	70	0	80	90
Steel aggregate	Punchings	Sheared bars	7.78	—	99	0	120	130
Steel shot	Chilled	SAE standard	—	7.50	98	0	120	130

*These aggregates are representative of some of the best which can be obtained commercially.
†This ore is primarily magnetite, with some hematite (Fe_2O_3) and limonite.
‡Grout sand will cost about $50, $40, and $30 per ton for limonite, barite, and magnetite, respectively. Values for processed aggregates include $10 per ton for freight.

TABLE 9—AVERAGE VALUES OF ABSORPTION FACTORS FOR CONCRETE AGGREGATES

Material	Chemical composition	Specific gravity	Absorption, sq cm per g	
			Fast neutrons*	Gamma rays (3 Mev)
Alumina	Al_2O_3	3.6	0.0330	0.0365
Barite	$BaSO_4$	4.3	0.0236	0.0363
Ferrophosphorus	Fe_nP	6.4	0.0230	0.0359
Goethite†	$Fe_2O_3 \cdot H_2O$	3.7	0.0372	0.0362
Iron	Fe	7.8	0.0214	0.0359
Lead	Pb	11.3	0.0103	0.0413
Lime	CaO	3.1	0.0285	0.0372
Limestone	$CaCO_3$	2.7	0.0327	0.0368
Marble	$CaCO_3$	2.7	0.0327	0.0368
Magnetite	Fe_3O_4	4.6	0.0258	0.0359
Quartz	SiO_2	2.6	0.0328	0.0362
Water	H_2O	1.0	0.10–0.134	0.0396

*Computed values using relationship: $0.0852\ A^{-1/3}$ sq cm per g for elements having atomic weight A, except for measured values given in Reference 10.
†Limonite having 10 percent water by weight.

aggregates such as ferrophosphorus and iron punchings should be considered only when it is necessary to obtain concrete weighing over 250 lb per cu ft. Densities as high as 425 lb per cu ft can be produced using steel punchings and iron shot. Limonite or goethite may be used in combination with steel punchings, and shot, or in combination with magnetite, in order to increase the fixed water content of the resulting concrete.

USE OF BORON IN CONCRETE

Boron compounds may be added to concrete to increase the probability of neutron capture without producing secondary gamma rays of high energy. Only small amounts of boron can be used due to its deleterious effects on strength and delay in set. The barite-boron concrete developed at Oak Ridge National Laboratory weighs about 200 lb per cu ft and contains from 0.2 to 1.0 percent boron by weight. Due to the high cost of boron, and the limited improvement in the attenuation properties caused by adding small amounts of boron to concrete, it is difficult to justify using boron except for special applications. If boron is required, it may be more desirable to use boral sheets or to apply a layer of boron mortar to the inside face of a shielding wall instead of adding boron to the concrete.

Of the available boron compounds, colemanite, boron frits, and ferroboron are of most interest for use in concrete. If colemanite is used, it should be added in the form of coarse sand and not powder. Concrete made with boron frits (formed by fritting borax with silica) will cost more, contain less water for the same boron content, but will be easier to formulate and cure.

Accelerators or high-alumina portland cement have been used so that concrete containing boron compounds will harden. In some cases, it is necessary to keep the concrete covered with water for several weeks.

PLACEMENT METHODS

Careful consideration should be given to placement methods in the design and construction of a high-density concrete shield. Consideration should be given to factors such as aggregates, density, composition of mix, quantity of concrete required, available equipment, complexity of forms, type and number of embedded items, placing conditions, and experience of workmen.

Conventional methods

In general, when the concrete does not contain pieces of steel as coarse aggregate, it can be mixed and placed successfully by conventional methods in which the aggregates, cement, and water are mixed together and then placed in the forms. Special equipment is not required; however, care should be taken to prevent overloading the mixer, and segregation of aggregates within the mix. Standard methods and equipment should be used when possible. Vibration during placement is beneficial but must be used with discretion. Ordinarily, conventional methods for placing concrete will be cheaper; therefore, when natural heavy aggregates are used and the forms are relatively free of embedded items, there will be little incentive to use the special methods described below.

Pumping methods

Conventionally mixed concrete made with heavy aggregates can be pumped from the mixer to the forms through specially constructed pipe lines. Special mixing and pumping equipment is available for pumping concrete of normal density. If allowance is made for increased density, this equipment can be used also to pump concrete made with dense natural aggregates. However, the pumping of concrete containing coarse steel aggregate is not recommended since pieces of steel tend to segregate or lodge in the pump or in the equipment. Usually, the maximum density that can be obtained by this method is considerably less than for concrete placed by other methods. Heavy grouts or mortars containing steel shot, fine ferrophosphorus, and fine sands of barite, magnetite, or limonite can be pumped satisfactorily.

Prepacked method

In the prepacked method coarse aggregates are placed in the forms and the interstices are later filled with a special grout. Several advantages over conventional methods for placing concrete under certain conditions are:

1. Segregation of coarse aggregates, especially steel punchings, can be minimized.
2. Concrete of uniform density and composition can be placed more easily in confined areas and around embedded items.
3. For similar materials, greater density and homogeneity can be obtained more consistently than by other methods.

Another advantage of the prepacked method is that several types of material can be used together as coarse aggregates, for example: magnetite and limonite, limonite and steel punchings, or magnetite and steel punchings. These combinations can be blended accurately and efficiently with commercial equipment

adapted to the particular job. In placing coarse aggregate in the forms, the type of coarse aggregate can be changed from area to area so that the best shielding material will be where it will do the most good. Three types of high-density concrete were placed in the Naval Research Reactor. On another job, a half-million dollars was saved by replacing steel punchings with magnetite in shield areas where the higher density was not required.

In general, the prepacked method assures that the heavy aggregate distribution is uniform throughout the shield and that no large air voids exist. However, there are also certain disadvantages of the prepacked method, namely: only a few companies are experienced in this type of construction; construction joints are difficult to execute and may present planes of weakness; and finishing the top of a large concrete area is more difficult and expensive. Placement costs will generally be somewhat greater for the prepacked method than for conventional methods.

Puddling methods

Another special method for placing high-density concrete consists of putting several inches of mortar in the form first and then covering the mortar with a layer of coarse metallic aggregate. The coarse aggregate is puddled into place by rodding or internal vibration. This method has several advantages, depending upon job conditions; one advantage is that the steel aggregate is not passed through the mixer. Adequate control measures are required to obtain the required density consistently throughout the wall. Pieces of steel aggregate cannot be moved horizontally to any great extent once they are lodged in the mortar. Thus it is often difficult to work aggregate beneath block-outs in the shield or around embedded items.

FORMWORK

Forms for concrete shields may be removable or permanent. For concrete shields that do not require close tolerance in alignment or do not contain numerous embedded items, removable forms can be relatively simple and inexpensive. Form costs will generally increase with the number of embedded items, close tolerances, density of the concrete, and complexity of shield construction. When a metal shield enclosure is required for other reasons, permanent forms of steel or aluminum may be used. Metal forms can be prefabricated or constructed entirely in place. If only a thin surface plate is required, it may be cheaper to use removable wood forms, and then attach the steel surface plate to the hardened concrete. Relative cost figures are given in Table 10 for different types of concrete forms. As shown in Table 11, the cost of elaborate steel forms may be greater than the high-density concrete.

TABLE 10—CONCRETE FORMWORK*

Item	Cost
Wood forming	
150 lb per cu ft concrete	$1.50 per sq ft
300 lb per cu ft concrete	$3.00 per sq ft
Steel forming	
½ in. thick plate	$3.00 per sq ft for large plate
Welding	
½-in. plate including cleaning and clamping	$3.00 per linear ft

*Chicago area (1956), data from E. E. Hamer, Argonne National Laboratory

TABLE 11—TYPICAL CONSTRUCTION COSTS FOR CONCRETE SHIELDS

Concrete	Type*	Type of forms†	Forms, dollars per cu yd	Concrete, dollars per cu yd	Total cost, cents per lb
Ordinary structure		Removable wood	45	25	1.7
Ordinary shielding	Q	Removable wood	60	40	2.4
High-density	K or N	Removable wood	100	120	3.7
High-density	O	Permanent steel	300	250	9.5
High-density	F	Permanent steel	600	500	15.5

*See Table 2 for a description of these concretes.
†The complexity of the formwork is assumed to increase as indicated by the corresponding cost figures. The increase in the cost of removable wood forms is related to the density of the concrete and the number and type of embedded items. Cost of permanent steel forms will also be greater for intricate design. In the bottom example, numerous tubes are located in a close lattice arrangement, typical of a charging face for a reactor. Removable wood forms can be used for any type of high-density concrete if the design permits.

Dimensional tolerances

For some types of reactor shields, close tolerances must be maintained in the alignment of tubes and equipment which penetrate the shield. Plywood forms with oversize holes can be aligned in the field to 1/32-in. tolerance using a transit. Portable steel jigs can be used in conjunction with plywood forms. However, a tolerance of plus or minus 1/16 in. is more reasonable. By using permanent steel forms with drilled holes, numerous tubes can be located to within 1/64 in.

Permanent steel forms may be required to support structural loads during and after construction, provide additional shielding, protect the concrete, act as a gas barrier, and provide a means for attaching equipment to the shield. However, permanent steel forms are expensive and their use should be considered carefully. Likewise, tolerances required for the work should be questioned and values specified which can be obtained with reasonable cost and effort in the field. Relaxation of close dimensional tolerances for the construction of shielding structures will result in substantial cost savings. Simplicity of shield design and construction should be sought to reduce costs and improve the over-all plant design.

CONCRETE BLOCK USED FOR SHIELDS

High-density concrete can be prefabricated into block of different shapes and sizes. Various types can be purchased directly from certain vendors. In general, the density of small blocks will be somewhat less than for large blocks made with the same aggregate since smaller pieces of aggregate are required for the smaller blocks. Dimensional tolerances are also important. The 1/8-in. variation common to concrete blocks is acceptable for 6 x 6 x 12-in. block since the length-thickness ratio for a crack is at least 48. For smaller block, cracks between adjacent bricks become appreciable so that dimensional tolerances should be 1/16 or 1/32 in. Mortar is recommended for constructing shields made with the smaller sizes of concrete block. Mortar of greater than normal density can be obtained by using magnetite or barite sand. If mortar is not desired, the wall should be made about 10 percent thicker than a solid wall.

Attention should be given to alternating and staggering courses so that cracks between bricks will not line up. However, the size of brick that can be handled by a workman is limited by weight and, there is a limit on the safe height-width ratio for a free-standing block wall. A heavy concrete block wall is usually more expensive than a monolithic structure of cast-in-place concrete. Therefore, the use of block made with high-density concrete is usually limited to special applications where movable sections are desired; for example, a temporary shield in a tunnel where occasional access is required.

Larger blocks weighing several tons, and handled by heavy equipment, can also be prefabricated to desired dimensional tolerances by any one of the methods described above, and cured carefully. Such large prefabricated blocks can be used as removable sections of a large shield, if desired. They will cost more than monolithic concrete shields.

SHIELD COST SUMMARY

It is difficult to estimate exactly what a radiation shield will cost; the data presented herein are offered with reservation and must be modified to meet local conditions. Table 8 gives procurement costs for heavy aggregates. Material costs for ingredients in various types of high-density concrete are included in Table 2. To these material costs must be added the cost of forms, manpower and equipment costs required to mix and place the concrete, and construction and administrative overheads. On one job using elaborate steel forms, the cost breakdown was as follows: 16 percent for form fabrication; 20 percent for materials other than heavy aggregate; 30 percent for labor at the construction site; 12 percent for equipment usage; 15 percent for steel aggregate; and 7 percent for magnetite and limonite aggregate. In this shield, limonite and magnetite occupied over 50 percent of the concrete volume. Cost data indicate that the greatest potential cost savings are in simplifying formwork, reducing labor at the site, and in using natural aggregates instead of more expensive metallic aggregate.

Table 11 shows typical cost data for different types of concrete shielding structures. These are direct construction costs based primarily upon costs for reactor shields. To obtain an indication of the *total cost* for a given shield, the figures in Table 11 should be increased about 20 to 35 percent to allow for project overheads. Note that by using iron or steel aggregates, concrete densities from 240 to 425 lb per cu ft can be obtained at a material cost of 2 to 6 cents per pound. Also, neutron shields will cost substantially more than gamma ray shields which do not require a high content of "fixed" water. However, concrete of such high density can be justified economically only when it is essential to minimize wall thicknesses. In locations where barite or magnetite is available at 1 to 1.5 cents per lb, high-density concrete weighing around 220 lb per cu ft is usually a judicious choice.

On the other hand, if space is not at a premium and shielding walls can be 8 or 10 ft thick, then conventional concrete weighing around 150 lb per cu ft

may be the logical choice. In some cases, natural earth and bedrock can be utilized effectively. To get the best shield design for a given project, a careful cost study of several layouts and building arrangements is recommended. The data in this paper will assist the engineer in performing these studies and in arriving at a feasible and economical shield design.

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Title No. 54-57

Tests of Full-Sized Prestressed Concrete Bridge Beams*

By INGE LYSE†

SYNOPSIS

Two full-sized prestressed concrete beams, identical with those made for Mandal Bridge in Norway, were tested to failure. The elastic and plastic deformations of the concrete due to prestressing and the shrinkage due to the drying of the concrete were observed. The load at first cracking agreed well with the computed value, and the strains measured in the beams agreed fairly well with the computed stresses. The load at failure corresponded with the load computed on the basis of the so-called "simplified theory."

INTRODUCTION

Fig. 1 shows span dimensions and general layout for the Mandal Bridge in south Norway. The bridge consists of six equal spans of 50.85 ft (15.5 m) and a main span of 68.90 ft (21.0 m). There are six prestressed concrete beams for each span.

To investigate the behavior of full-sized bridge beams, the Bridge Section of the Norwegian Highway Department requested the Concrete Laboratory of Norway's Institute of Technology to assist in carrying out tests to destruction of two of the prestressed concrete beams.

MATERIALS

The prestressing wire, *Neptunstahl* type from Germany, has a rectangular cross section with nominal dimensions of 3 x 6 mm (0.118 x 0.236 in.) with small deformations and twisted about its axis. Because of this twisting it was suspected that a certain amount of internal stresses might be present, and that these might effect the stress-strain relationship as well as the elongation at failure. Actually, little effect was noted in the stress-strain relationship while the elongation at failure was only about 2 percent measured on a gage length of 3.28 ft (1.0 m). The average modulus of elasticity of the wires was found to be 29,400,000 psi, yield point about 190,000 psi, and ultimate strength about 234,000 psi. There was a surprisingly large variation in the ultimate strength of the wires. A total of 126 wires were tested and gave a maximum variation of about ± 13 percent. With such a large variation in the wires used for prestressing it would be expected that the ultimate load of the beams would vary considerably.

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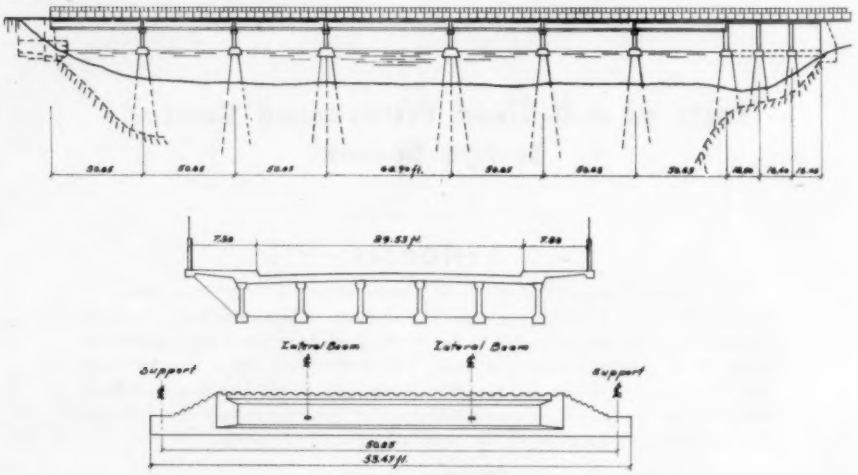


Fig. 1—Prestressed concrete bridge at Mandal, Norway

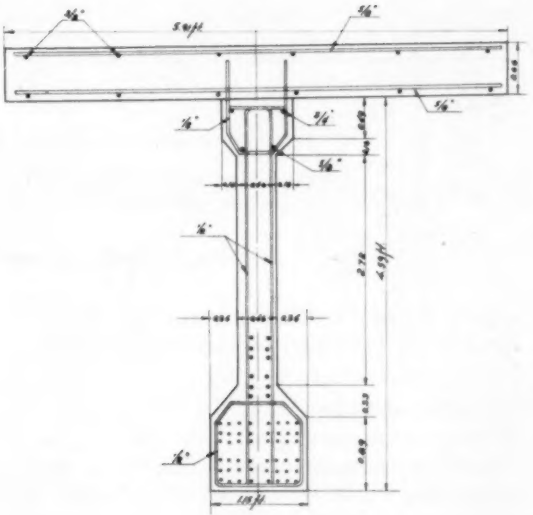
High-early-strength cement was used in the concrete for the beams. Eleven days of moist curing was considered sufficient for the concrete before the pretensioned wires were cut. At this age the average strength of the concrete as measured on 6 x 12-in. cylinders was about 5700 psi (400 kg per sq cm). The two test beams were cured in air for another 40 days before the top slab was cast. Fig. 1 shows the beam before the top slab was cast and Fig. 2 the details of the beam with the top slab.

The beams were given another 11 days of moist curing after casting the top slab and then remained in the air of the production plant until they reached the age of 92 days when they were tested to failure. The concrete control specimens showed little increase in strength between the ages of 11 and 92 days. The secant modulus of elasticity of the concrete was determined on 6 x 12-in. cylinders and averaged 3,500,000 psi.

PRESTRESSED CONCRETE BEAMS

The design of the beams used in the bridge is shown in Fig. 1 and 2. The beams were 53.47 ft long, 4.59 ft deep, and the top slab was 0.66 ft thick. They were cast and prestressed at the bridge site. The top section of the beam itself is of a stepped-off shape as shown in Fig. 1. This was done to provide proper resistance to the horizontal shearing stresses along the juncture of the beam and the slab. These stresses are of particular importance in the region of the supports. The beams were of rectangular cross section in the regions of the supports, while the major part of the beams consisted of an I-section (Fig. 2). In addition to the prestressing wires a considerable amount of ordinary reinforcing bars were placed in the beam as well as in the top slab. The thickness of the slab corresponded to the thickness of the bridge slab and was 7.9 in. The slab was not prestressed.

Fig. 2—Cross section of test beams



The wires for the beams were pretensioned in a fixture consisting of heavy structural steel sections with rigid end arrangements. Tensioning was done by jacks and both the elongation of wires and the oil pressure in the jacks were measured. To secure as even a stress as possible in the 60 wires used in each beam the wires were tensioned in two steps. The wires were tensioned to a stress of 120,000 psi, which corresponded to 63 percent of the yield point stress. The total prestressing force of the 60 wires was therefore about:

$$F = 120{,}000 \times 0.0278 \times 60 = 200{,}000 \text{ lb}$$

This load would give a maximum compressive stress at the bottom of the beam of about 1065 psi and a tensile stress at the top of the beam of 285 psi. As the load was transferred to the concrete at the time of cutting the wires an elastic deformation resulted. During the 81 days between the cutting of the wires and the testing of the beams shrinkage and creep added to the deformation.

To investigate the amount of shrinkage during this period special concrete slabs were made. Observations of these slabs indicated a shrinkage of about 0.0072 percent during the 81 days.

Strain gage measurements on the prestressed wires showed a decrease in the strain of about 6 percent, indicating that the creep was about twice as large as the observed shrinkage.

TESTING ARRANGEMENT

The testing arrangement is shown in Fig. 3. Two steel beams were used as the loading base. At the supports, tie bars transmitted the load from the concrete beam to the steel beams; at 11.75 ft from each support, oil jacks

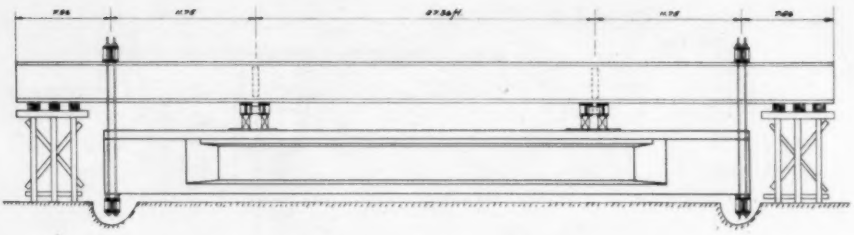


Fig. 3—Testing arrangement

applied the load to the test beam. By this two-point loading arrangement the shear in the test beam became relatively large as compared with the moment.

Oil pressure in the loading jacks was measured by two manometers which had been calibrated before the testing and which were recalibrated as soon as the tests were completed. An extensive investigation of deformations and deflections was made of the beams during the loading tests. The strains in the prestressing wires were observed by means of Philips GM 4473 strain gages, which are similar to the Baldwin SR-4 strain gages. The strain gages were glued to the twisted wires, but since this was a difficult procedure some of the gages gave erratic results.

Deformations in the concrete were observed by means of ordinary extensometers of 8 in. gage length. The results were very satisfactory.

The center deflections of the beams were measured by accurate level observations.

CRACKING OF THE CONCRETE

The compressive stress in the concrete at the bottom of the beams was calculated to be about 1065 psi at the time of cutting the pretensioned wires. Because of shrinkage and creep during the time between the cutting of the pretensioned wires and the loading of the test beams a stress reduction of about 175 psi resulted. The actual compressive stress in the concrete at the time of test was therefore about 890 psi. During the testing the compressive stress in the concrete at the bottom of the beam decreased and reached zero at an external moment of 780,000 ft-lb. With increased loading the tensile stress in the concrete increased and at an external moment of about 1,300,000 ft-lb the strain measurement as well as the discontinuity in observed center deflection indicated that the first cracking occurred in the concrete. Visual inspection also showed that very fine vertical cracks appeared at the center portion of the beams. The cracks extended from the bottom of the beam into the stem of the beam.

At the time of cracking the maximum tensile stress in the concrete at the bottom of the beams was computed to be about 600 psi. The computation was made on the assumption of uncracked transformed section: $\sigma = (M/I)c$. As the compressive strength of the concrete was about 5700 psi at the time of

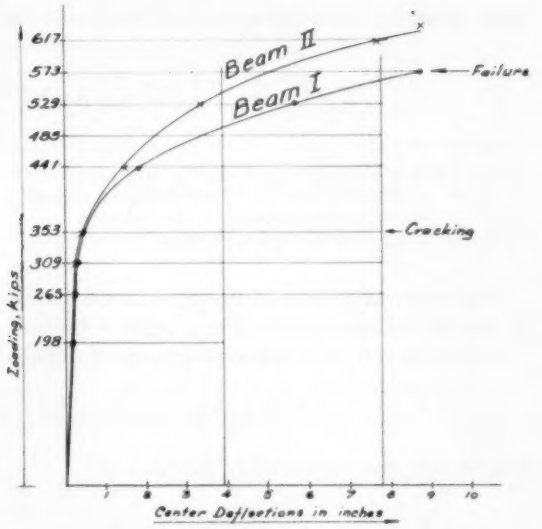


Fig. 4—Center deflections of test beams

test, the tensile stress of 600 psi at time of cracking seems reasonable. [If computed by ACI-ASCE Committee 323 method,* $f_t' = 7.5\sqrt{f_c'} = 7.5\sqrt{5700} = 565$ psi (about 6 percent lower).]

The amount of cracking of the concrete increased as the load increased so that near the ultimate load the cracking pattern covered the whole stem of the beams. The directions of the cracks followed closely the directions of the principal stresses computed for the various sections of the beams.

Fig. 4 shows the center deflections of the two beams during testing. Up to the load at which cracking of the concrete took place the deflections were nearly the same for the two beams. In Beam 1 a loud report was noted at the cracking load indicating that one or more of the 60 prestressed wires might have ruptures. At higher loads the center deflections of Beam 1 were considerably greater than for Beam 2.

FAILURE OF BEAMS

The beams failed by rupture of the prestressed wires. The cracks in the beams extended into the top slabs as the loading approached the ultimate values. Beam 1 ruptured at a moment of about 1,910,000 ft-lb and Beam 2 at a moment of about 2,036,000 ft-lb.

Using the Suenson method† of computing the failure of ordinary reinforced concrete beams (where the yielding of the reinforcement is reached before the compressive stress in the concrete reaches the ultimate strength) we have,

*ACI-ASCE Committee 323, "Tentative Recommendations for Prestressed Concrete," Section 207.3.3, ACI JOURNAL, Jan. 1958, *Proc.* V. 54, p. 555.

†Suenson, E., "Jaernprocentens Indflydelse paa Jaernbetonpladers Baereevne," *Ingenigren* (Copenhagen), V. 21, 1912, p. 568.

when assuming full development of the tensile strength of the prestressed wires:

$$T_s\left[h_s - \frac{T_s}{2f_c'b}\right] = M_{ult}$$

where:

T_s = tensile strength of the 60 wires
h_s = distance from top of slab to center of gravity of prestressed wires
f_c' = compressive strength of the concrete
b = width of top slab = 5.91 ft

The center of gravity of the prestressed wires was 0.65 ft from the bottom of the beams. Consequently: $h_s = 5.25 - 0.65 = 4.60$ ft.

Assuming full ultimate strength of all wires at the time of failure we get:

$$T_s = 234{,}000 \times 0.0278 \times 60 = 390{,}000 \text{ lb}$$

This should give an ultimate moment of:

$$390{,}000\left(4.60 - \frac{390{,}000}{2 \times 5700 \times 5.91 \times 12^2}\right) = 390{,}000\,(4.60 - 0.04) = 1{,}780{,}000 \text{ ft-lb}$$

The method of calculating ultimate strength of bonded prestressed concrete members given by ACI-ASCE Committee 323 in its 1958 method* for a section of the type used in the beams is:

$$M_u = A_s f_{su}\, d\left(1 - \frac{k_2}{k_1 k_2}\frac{pf_{su}}{f_c'}\right)$$

or this may also be written as

$$M_u = A_s f_s'\, d\left(1 - 0.60\frac{p\, f_s'}{f_c'}\right)$$

For the test beams we have:

$$M_u = 1.668 \times 234{,}000 \times 4.60\left(1 - 0.60\frac{1.668 \times 234{,}000}{144 \times 27.186 \times 5700}\right)$$

$$= 1{,}790{,}000\,(1 - 0.01) = 1{,}772{,}000 \text{ ft-lb}$$

which is practically the same as by the Suenson method.

The observed moments at failure were 1,910,000 and 2,036,000 ft-lb, respectively, for the two beams, indicating that at the section of failure the total strength of the wires was 7.3 and 14.4 percent greater than the sum of the average strengths of the wires tested. With the considerable variation found

*ACI-ASCE Committee 323, "Tentative Recommendations for Prestressed Concrete," Section 209.2.1 (a), ACI JOURNAL, Jan. 1958. *Proc.* V. 54, p. 558.

in the wires tested in lengths of 27 in., the agreement between calculated and observed ultimate moments must be considered very good.

CONCLUSIONS

The following conclusions may be made on the basis of the observed results.

1. The bridge beams were manufactured satisfactorily at the construction site.

2. The wires used for prestressing showed considerable variation in ultimate strengths.

3. The effect of the shrinkage and creep of the concrete between the time of cutting the pretensioned wires and the testing to failure of the beams (81 days) was small, only about a 6 percent decrease in prestressing.

4. Proposed methods of predicting cracking load and ultimate load in the ACI-ASCE Committee 323 recommendations gave satisfactory, slightly conservative results.

5. Composite action between precast and cast-in-place elements in the actual bridge is performing as designed.

Discussion of this paper should reach ACI headquarters in triplicate by Aug. 1, 1958, for publication in the Part 2, December 1958 JOURNAL.

Title No. 54-58

Concept of Elastic Parameters*

By VALERIAN LEONTOVICH†

SYNOPSIS

The concept of elastic parameters, defining mathematical relations between elastic properties of the member and its elastic parameters, is presented. It is further shown that integration of the concept into the elastic-center method substantially shortens and simplifies analysis of frames and arches of variable cross section. So that the concept and its application may be fully appreciated, a number of examples are presented.

INTRODUCTION

The concept of elastic parameters is introduced in this paper as a new interpretation and extension of the moment-area principles. It is shown that various elastic properties of straight and curved nonprismatic members may be directly expressed in terms of defined elastic constants, as parameters and a dimensional multiplier.

The concept of elastic parameters has wide practical application. It offers immediate and simple solutions to many hitherto complicated problems. Among its diverse applications, the integration of the concept into the elastic-center method for frames and arches of variable cross section may be considered a major practical application. Therefore, this paper centers primarily on the explicit presentation of the concept of elastic parameters and its integration into the elastic-center method.

The elastic-center method has the advantages of being a direct method of solution and of being uninvolved in its application. Its extreme utility has been demonstrated in application to the analysis of frames suffering "body swing" under load. The great disadvantage of the elastic-center method is that it is inherently time consuming and tedious. Examination of a simple example will demonstrate this.

Consider a frame with nonprismatic members, which is symmetrical about its vertical center line. Analysis by the elastic-center method would follow the four-step procedure outlined on the following page.

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1. The elastic center of the structure is obtained from the familiar equation

$$y_o = \frac{\int_0^s \frac{y'\,ds}{EI}}{\int_0^s \frac{ds}{EI}} \qquad (1)$$

Both numerator and denominator of the expression must be evaluated by semigraphical integration.

2. The moments of inertia of the elastic weight of the frame are obtained about the principal axes. Again semigraphical integration is required for evaluation of the following:

$$I_x = \int_0^s \frac{y^2\,ds}{EI} \qquad (2)$$

$$I_y = \int_0^s \frac{x^2\,ds}{EI} \qquad (3)$$

3. The moments of the area of the M/EI diagram about the principal axes of the frame are determined. This step also requires a semigraphical procedure.

4. Statically indeterminate forces may then be calculated by established formulas.

Each step involving semigraphical integration requires that the members of the frame be subdivided into elements and that calculations be carried out upon each element. The major portion of time and labor expended in analysis by the elastic center method is presently consumed in obtaining elastic properties of the frame by this method.

This paper will conclusively demonstrate that, through the use of the concept of elastic parameters, the analysis of statically indeterminate frames with straight or curved nonprismatic members may be accomplished in substantially the same manner as that of frames containing prismatic members alone.

SIGN CONVENTIONS AND NOTATION

The familiar sign convention is used; that is, moments producing compression in the upper fibers are denoted as positive, and those producing tension in upper fibers are denoted as negative. Moment diagrams are always drawn on the tension side of the member.

The sign convention for angular rotation of a member's axis is coordinated with the bending moment convention. Accordingly, the direction of angular rotation is considered positive when it coincides with the moment's positive direction.

The axis of a straight member of variable cross section is assumed in this paper as the line parallel to the straight edge of the member, and passing through the center of gravity of the smallest cross section.

In addition, it should be noted that the term *elastic weight of the member* is extensively used in this development, and means the fictitious mass represented by $\int ds/EI$, when the integration is extended over the entire length of the member.

Notation

The letter symbols introduced in this paper are defined where they first appear, in the text or illustration. The important symbols are:

$\left.\begin{matrix}\alpha_n \\ \alpha_n + 1 \\ \beta_n\end{matrix}\right\}$ = elastic parameters of member $(n) - (n + 1)$

γ and δ = elastic parameters of curved member

φ = angle of the slope of the member's axis

Θ_n = elastic constant

E = modulus of elasticity

f = rise of an arched member

H = statically indeterminate quantity

I = moment of inertia

I_o = reference moment of inertia of member (I of member's minimum cross section)

K = projected length of member's axis on X-axis

L = length of straight line connecting terminal points of member's axis; also span of frame or arch

M = bending moment

M_o = static bending moment at an arbitrary section

R = load constant of straight number

s = length of the member's axis

S and T = load constants of curved member

U = load constant of curved member; also energy of flexural deformation

V and Z = statically indeterminate quantities

ELASTIC PARAMETERS

To facilitate analysis of structures with members of variable cross section, the use of elastic constants has been introduced elsewhere for certain methods. For straight members there are available many comprehensive tables and charts expressing these constants in various systems. Each system of constants is originally developed to accommodate and to serve a particular method of analysis. The utility of certain elastic constants is, however, far beyond the scope of their original objectives.

Among the various systems expressing shape characteristics and load constants for members of variable cross section, the one featuring angular deformations of the simply supported member is perhaps the most fundamental and promising. This paper demonstrates that physical properties of the straight or curved member of variable cross section may be precisely and simply defined through the use of the constants of this system. Some properties of a member are expressed by individual elastic constants, other expressed as functions of two or three constants. These constants are herein termed *elastic parameters*, since they may be used parametrically in defining the elastic properties of the member. The relations thereby expressed are compiled as the *concept of elastic parameters.*

Before the physical interpretation of the elastic parameters is given and its significance emphasized, the manner of their derivation and definition should be first explained.

Elastic parameters may be derived from angles of rotation of the ends of a simply supported member when the latter is subjected to the action of an applied moment.* In the following derivations, the in.-lb dimensional system is employed.

Referring to Fig. 1, and the general member 1-2, oriented as shown in the X-Y Cartesian coordinate system, the angles of rotation α^0_{12} and β^0_{21} of the left and right ends of the member, respectively, due to a 1 lb-in. moment applied at the left end may be expressed as follows:

$$\left.\begin{aligned} \alpha^0_{12} &= \int_0^s (1 \text{ lb-in.}) \frac{(K - x)^2\, ds}{EIK^2} \\ \beta^0_{21} &= \int_0^s (1 \text{ lb-in.}) \frac{(K - x)x\, ds}{EIK^2} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (4)$$

where s denotes the length of the member along its axis, and ds is the differential of the stated length.

Introducing a reference moment of inertia I_o, which is taken as the I of the minimum cross section, and recognizing K, E, and I_o as constants, both parts of Eq. (4) may be rewritten in terms of dx as follows:

$$\left.\begin{aligned} \alpha^0_{12} &= \frac{1}{EI_oK^2} \int_0^K (1 \text{ lb-in.}) \frac{I_o}{I} \frac{(K - x)^2\, dx}{\cos \varphi} \\ \beta^0_{21} &= \frac{1}{EI_oK^2} \int_0^K (1 \text{ lb-in.}) \frac{I_o}{I} \frac{(K - x)x\, dx}{\cos \varphi} \end{aligned}\right\} \quad \ldots\ldots\ldots (4a)$$

Similarly, the angles of rotation of the right and left ends of the member, respectively, due to a unit moment applied at the right end may be expressed as follows:

$$\left.\begin{aligned} \alpha^0_{21} &= \frac{1}{EI_oK^2} \int_0^K (1 \text{ lb-in.}) \frac{I_o}{I} \frac{x^2\, dx}{\cos \varphi} \\ \beta^0_{12} &= \frac{1}{EI_oK^2} \int_0^K (1 \text{ lb-in.}) \frac{I_o}{I} \frac{(K - x)x\, dx}{\cos \varphi} \end{aligned}\right\} \quad \ldots\ldots\ldots (5)$$

*Original derivation of the elastic constants is due to A. Strassner. See his *Neuere Methoden zur Statik der Rahmentragwerke und der elastischen Bogentrager*, Wilhelm Ernst und Sohn, Berlin, 1921. See also Ernst Suter, *Die Methode der Festpunkte*, Julius Springer, Berlin, 1932. The writer's version presented herein, however, is more general and applicable to members of any shape and curvature.

Elastic parameters α_{12}, α_{21}, β_{12} and β_{21} now may be defined as numerical quantities derived from the angles of rotation:

$$\left.\begin{aligned} \alpha_{12} &= \alpha^0_{12}\,\frac{EI_o}{K\,(1\text{ lb-in.})} \\ \alpha_{21} &= \alpha^0_{21}\,\frac{EI_o}{K\,(1\text{ lb-in.})} \\ \beta_{12} = \beta_{21} &= \beta^0_{12}\,\frac{EI_o}{K\,(1\text{ lb-in.})} \end{aligned}\right\} \qquad \text{.....(6)}$$

After cancellation of common terms in the numerators and denominators, the equations of the elastic parameters, with respect to the X-axis, are

$$\left.\begin{aligned} \alpha_{12} &= \frac{1}{K^3}\int_0^K \frac{I_o}{I}\,\frac{(K-x)^2\,dx}{\cos\varphi} \\ \alpha_{21} &= \frac{1}{K^3}\int_0^K \frac{I_o}{I}\,\frac{x^2\,dx}{\cos\varphi} \\ \beta_{12} = \beta_{21} &= \frac{1}{K^3}\int_0^K \frac{I_o}{I}\,\frac{(K-x)x\,dx}{\cos\varphi} \end{aligned}\right\} \qquad \text{.....(6a)}$$

CONCEPT OF ELASTIC PARAMETERS

As stated previously, various elastic properties of a straight or curved member may be defined in the terms of the elastic parameters of the member and a dimensional multiplier. Thus, for example, the location of the elastic center of a member, its moments of inertia about certain axes, and many other properties may be precisely defined.

Since analysis of these relationships involves an extended interpretation of the moment-area principles, the presented development may be regarded as an extension of these principles. It is therefore presented in the form of nine corollaries to the familiar moment-area theorems. Expressed in general form, and formulated with respect to the X-axis, for a curved nonprismatic member 1-2, with supports on different levels, as shown in Fig. 1, they serve to define the abovementioned relations.

Corollary 1

For any member with simply supported ends on different levels, the elastic weight of the member may be expressed as a product of $(\alpha_{12} + \alpha_{21} + 2\beta_{12})/\mathrm{EI}_o$ *and the projected length of the member on the* X*-axis.*

Introducing a convenient expression,

$$\Theta_{12} = \alpha_{12} + \alpha_{21} + 2\beta_{12} \qquad \text{.....(7)}$$

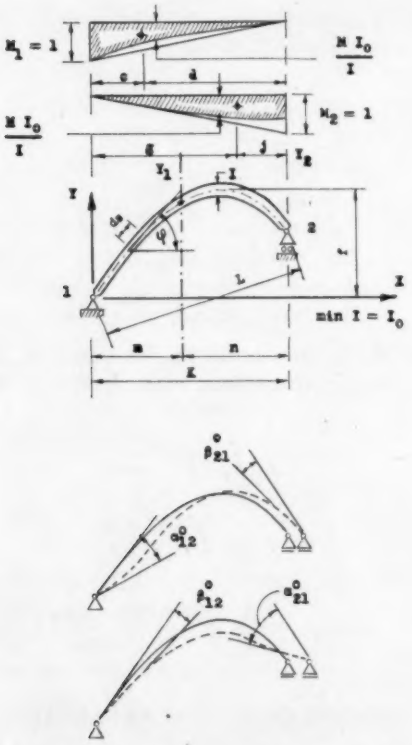

Fig. 1—Relationships between the area of M/EI diagram and the angular rotations of the curved member

the corollary may be expressed mathematically by the following equation:

$$\frac{1}{EI_o}\int_0^K \frac{I_o}{I}\frac{dx}{\cos\varphi} = \Theta_{12}\frac{K}{EI_o} \qquad \text{.....(8)}$$

In the special case that the supports of the member are on the same level, $K = L$, and Eq. (8) becomes

$$\frac{1}{EI_o}\int_0^L \frac{I_o}{I}\frac{dx}{\cos\varphi} = \Theta_{12}\frac{L}{EI_o} \qquad \text{.....(9)}$$

The proof of the above corollary follows from the relations between elastic parameters and the angles of rotation. The right part of Eq. (8) may be rewritten in terms of angles of rotation, through the use of Eq. (6), as

$$[(\alpha^o_{12} + \beta^o_{12}) + (\alpha^o_{21} + \beta^o_{21})]/(1 \text{ lb-in.})$$

which may be recognized as the fundamental expression

$$\frac{1}{(1 \text{ lb-in.})} \int \frac{M ds}{EI}$$

when M is equal to unity and the integration is extended along the entire length of the member's axis. Hence, it may be further rewritten as

$$\int \frac{ds}{EI}$$

and upon substitution of $dx/\cos \varphi$ for ds, the identity with the left part of Eq. (8) is obtained.

Because the elastic weight of any given member is constant, the mathematical statement of Corollary 1, leads to the important conclusion which, being formulated for an arbitrary member with simply supported ends and oriented in the Cartesian coordinate system, may be stated as follows:

There is only one value of the term θ_{12} for the two conjugate sets of elastic parameters (with three parameters per set) which define the elastic properties of the member in the direction of both coordinate axes; provided, however, that all parameters have been derived with respect to any one particular axis of the coordinate system.

Corollary 2

For any member with simply supported ends on different levels, the distances m *and* n *along the* X*-axis between the elastic center and the left and right ends of the member, respectively, may be expressed by the equations*

$$m = \frac{\alpha_{21} + \beta_{12}}{\theta_{12}} K \qquad (10)$$

and

$$n = \frac{\alpha_{12} + \beta_{12}}{\theta_{12}} K \qquad (11)$$

The proof of the first part of the above corollary follows from the relations between the elastic weight and the elastic parameters. Since the elastic weight of a member of the unit length may be expressed by θ_{12}/EI_o, and the reaction of the elastic weight at the right end (support) may be expressed by $(\alpha_{21} + \beta_{21})/EI_o$, then it follows that the elastic center of the member is located at a distance $[(\alpha_{21} + \beta_{21})/\theta_{12}] K$ from the left end of the member, measured along the X-axis.

Proof of the second part of the corollary may be obtained by inference.

Corollary 3

For any member with simply supported ends on different levels, the moments of inertia of the elastic weight about the Y- *and* Y_2*-axes may be expressed by the equations*

$$I_y = \alpha_{21} \frac{K^3}{EI_o} \qquad (12)$$

and

$$I_{y_2} = \alpha_{12} \frac{K^3}{EI_o} \qquad (13)$$

In the special case that the supports of the member are on the same level, L replaces K, in Eq. (12) and (13).

The proof of the first part of the above corollary lies in the significance of the expression α_{21}/EI_o and fundamentals of moments of inertia. From Eq. (5) and (6), it is evident that the expression α_{21}/EI_o represents the moment of inertia about the Y-axis of the elastic weight of the member whose X-axis projection is unity.

Consequently, for K, the projected length of the member, the expression for moment of inertia of the elastic weight, in absolute dimensional units become

$$I_y = \frac{\alpha_{21} K^2}{EI_o}$$

Similar proof may be derived for the second part of the corollary.

Corollary 4

For any member with simply supported ends on different levels, the moment of inertia of the elastic weight of the member about Y_1*-axis passing through the elastic center of the member may be expressed by the equation*

$$I_{y_1} = \left[\alpha_{21} - \frac{(\alpha_{21} + \beta_{12})^2}{\Theta_{12}}\right] \frac{K^3}{EI_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

In the special case where supports of the member are on the same level and the member is symmetrical about its center line, Eq. (14) reduces to

$$I_{y_1} = \frac{\alpha_{21} - \beta_{12}}{2} \cdot \frac{L^3}{EI_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

Examination of Eq. (14) and comparison with Eq. (8), (10), and (12) reveals that it is analogous to the familiar parallel-axis theorem for moment of inertia:

$$I_{cg} = I_y - \text{area} \times (\text{distance to centroid})^2$$

Corollary 4 may alternatively be formulated in terms of the parameter α_{12} instead of α_{21}.

Corollary 5

When a unit moment is applied alone to the left end of a member with simply supported ends on different levels, the area A *of the* M/EI *diagram may be expressed by the equation*

$$A = (\alpha_{12} + \beta_{12}) \frac{K}{EI_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

Similarly, *when a unit moment is applied alone to the right end, the area* A′ *may be defined as*

$$A' = (\alpha_{21} + \beta_{12}) \frac{K}{EI_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

The proof of this corollary becomes evident upon examination of Eq. (16). Since $\beta_{12} = \beta_{21}$, then α_{12}/EI_o and β_{12}/EI_o are the respective reactions at the right and left ends of the member of unit length when the M/EI_o area is considered as a load. The sum of the above reactions times K, the projected length of the member, then represents the area of the M/EI_o diagram for a general member. A similar argument applies to Eq. (17).

Corollary 6

When a unit moment is applied alone to the left end of any member with simply supported ends on different levels, the distances c *and* d, *along the* X-*axis, between the center of gravity of the area of the* M/EI *diagram and the* Y- *and* Y_2-*axes, respectively, may be expressed by the equations*

$$c = \frac{\beta_{12}}{\alpha_{12} + \beta_{12}} K \quad \ldots\ldots (18)$$

and

$$d = \frac{\alpha_{12}}{\alpha_{12} + \beta_{12}} K \quad \ldots\ldots (19)$$

The proof of this corollary is analogous to that presented for Corollary 2.

Corollary 7

When a unit moment is applied alone to the right end of any member with simply supported ends on different levels, the distances g *and* j, *along the* X-*axis, between the center of gravity of the area of the* M/EI *diagram and the* Y- *and* Y_2-*axes, respectively, may be expressed by the equations*

$$g = \frac{\alpha_{21}}{\alpha_{21} + \beta_{12}} K \quad \ldots\ldots (20)$$

and

$$j = \frac{\beta_{12}}{\alpha_{21} + \beta_{12}} K \quad \ldots\ldots (21)$$

Proof of this corollary becomes evident from previous discussion.

Corollary 8

When a unit moment is applied alone to the left end of any member with simply supported ends on different levels, the moments of the area of the M/EI *diagram about the* Y- *and* Y_2-*axes may be expressed by the equations*

$$M_y = \beta_{12} \frac{K^2}{EI_o} \quad \ldots\ldots (22)$$

and

$$M_{y_2} = \alpha_{12} \frac{K^2}{EI_o} \quad \ldots\ldots (23)$$

The proof of the above corollary is evident from inspection of Eq. (16) and (18).

Corollary 9

When a unit moment is applied alone to the right end of any member with simply supported ends on different levels, the moments of the area of the M/EI *diagram about the* Y- *and* Y_2*-axes may be expressed by the following equations*

$$M_y = \alpha_{21} \frac{K^2}{EI_o} \quad \text{(24a)}$$

and

$$M_{y_2} = \beta_{12} \frac{K^2}{EI_o} \quad \text{(24b)}$$

The proof of this corollary becomes evident from previous discussion.

In addition to the preceding corollaries an important statement should be made pertaining to straight inclined members:

The elastic properties of a straight inclined member may be always expressed in terms of the elastic parameters of the same member horizontally oriented.

As an example, Eq. (12) of Corollary 3 may be written for a straight member with supports on different levels in the following form:

$$I_y = \alpha'_{21} \frac{K^2 L}{EI_o} \quad \text{(25)}$$

where α'_{21} is the elastic parameter derived for same member having supports on same level.

LOAD CONSTANTS

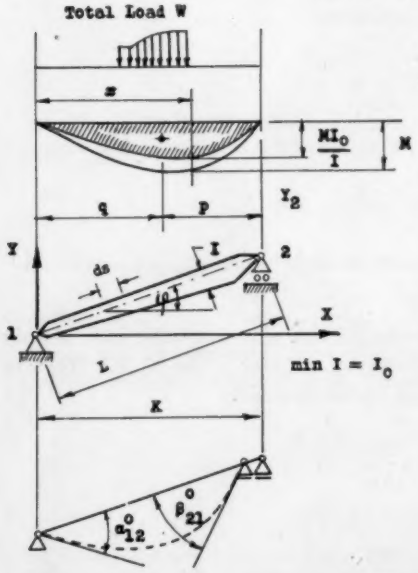

Fig. 2—Relationships between the area of *M/EI* diagram and the angular rotations of the loaded straight member

It has been shown that elastic parameters may be conveniently derived from the angles of rotation of a member's axis, when the member is subjected to action of a unit moment applied at the member's end. The same principle may be extended to obtain load constants of a member which is subjected to action of a moment induced by an arbitrary load on the member. By definition, these load constants are the constants which define the properties of the area of *M/EI* diagram. Applying a particular load to a member, and using the relationships outlined by Eq. (4) through (6a), the load constants for the particular loading condition may be obtained from the angles of rotation.

Thus, with reference to Fig. 2, and a straight* inclined general member 1-2, whose axis slope is $\cos \varphi = K/L$, the mathematical expressions of load constants are:

$$R_{12} = \frac{L}{(WK)K^3} \int_0^K \frac{I_o}{I} M(K - x)\, dx \quad \text{.....(26a)}$$

and

$$R_{21} = \frac{L}{(WK)K^3} \int_0^K \frac{I_o}{I} Mx\, dx \quad \text{.....(26b)}$$

where W defines a total load.

Since the expressions for the load constants were derived in the same manner as were those for the elastic parameters, although considering a moment diagram of different shape, the load constants may be regarded as a more general case of the elastic parameters. As a consequence, Corollaries 5-9 of the concept of elastic parameters are fully applicable to the load constants.

Furthermore, the previously described relations between the elastic constants of inclined and horizontally oriented members are also common to the load constants of the above-mentioned members.

APPLICATION TO THE ANALYSIS OF FRAMES WITH STRAIGHT MEMBERS

General

As stated previously the concept of elastic parameters permits extension of the theory of the elastic-center method,† thereby providing a short cut to the analysis of statically indeterminate frames with members of variable cross section. The applications and advantages are most readily explained by an example.

Numerical values of the elastic parameters and load constants for all members employed in the following example are taken from tables presented by Guldan.‡ The ft-kip dimensional system is used.

Example

Analyze a symmetrical portal frame, fixed at its supports and loaded with a concentrated load P of 10 kips as shown in Fig. 3. Consider the effect of the live load only. The width of all members normal to the plane of drawing is 1.5 ft.

The analysis begins with the selection of appropriate elastic parameters and load constants. In view of symmetry of the structure about its center line, only members 1-2 and 2-3 need be considered. These members are isolated as shown in Fig. 4, and for their respective shapes and proportions the elastic parameters are found to be as follows:

$$\alpha_{12} = 0.193 \qquad \alpha_{21} = 0.0682 \qquad \beta_{12} = 0.0572$$

$$\alpha_{23} = \alpha_{32} = 0.2437 \text{ and } \beta_{23} = 0.1525$$

*For an arbitrary curved member, the derivation of load constants is given on p. 1003.

†The writer prefers the method generally referred to as the "elastic-center" (neutral point) method, which was introduced by Otto Mohr and expanded by Muller-Breslau. See: Mohr, O., *Zeitschrift des Architekten und Ingenieur Vereins zu Hannover*, V. 27, 1881; and Muller-Breslau, H., *Die Neueren Methoden der Festigkeitslehre*, Baumgartner, Leipzig, 1886.

‡Guldan, R., *Rahmentragwerke und Durchlauftrager*, Springer-Verlag, Vienna, 1943, pp. 322-351.

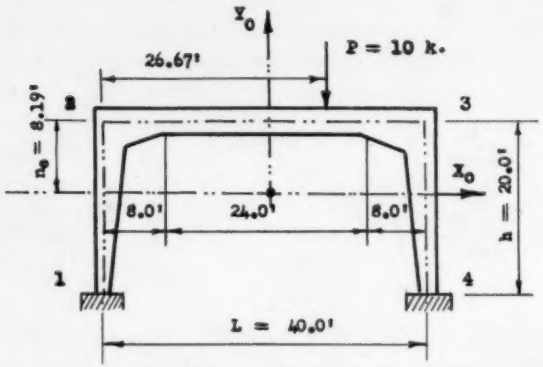

Fig. 3—Principal dimensions of the portal frame

For the left end of the girder, the load constant denoted by R_{23} is found to be 0.0464. Similarly, for the right end the load constant R_{32} is 0.0566. The ratio ϕ of reference moments of inertia of the members 1-2 and 2-3 is found to be as follows:

$$\phi = \frac{\text{minimum } I_{1-2}}{\text{minimum } I_{2-3}} = \frac{2^3}{3^3} = 0.297$$

Then, the elastic weight of the whole frame, denoted by B/minimum I_{1-2}, may be written in accordance with Corollary 1 as follows:*

$$\frac{B}{\text{minimum } I_{1-2}} = \frac{\phi L \Theta_{23}}{\text{minimum } I_{1-2}} + \frac{2h\Theta_{12}}{\text{minimum } I_{1-2}} \quad \ldots\ldots\ldots\ldots (27)$$

Substituting numerical values,

$$\frac{B}{\text{minimum } I_{1-2}} = \frac{0.297 \times 40 \times 0.7924}{\text{minimum } I_{1-2}} + \frac{2 \times 20 \times 0.3756}{\text{minimum } I_{1-2}}$$

$$= \frac{24.43}{\text{minimum } I_{1-2}}$$

The distance between the elastic center of the frame and the girder's axis may be determined from Eq. (28).

$$n_0 = \frac{2h \cdot h(\alpha_{12} + \beta_{12})}{2h\Theta_{12} + \phi L \Theta_{23}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Substituting numerical values

$$n_0 = \frac{2 \times 20^2 (0.193 + 0.0572)}{2 \times 20 \times 0.3756 + 0.297 \times 40 \times 0.7924} = 8.19 \text{ ft}$$

The moment of inertia of the elastic weight of the frame about X_0-axis passing through the elastic center of the frame may be expressed by the following equation:

$$I_{x_0} = \frac{2h^3 \alpha_{12}}{\text{minimum } I_{1-2}} - \frac{B n_0^2}{\text{minimum } I_{1-2}} \quad \ldots\ldots\ldots\ldots\ldots (29)$$

*Since the modulus of elasticity E is a common term that eventually cancels out in the process of determining the statically indeterminate quantities, it has been omitted from all elastic equations.

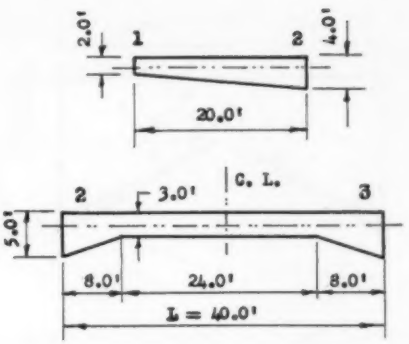

Fig. 4—Shape and dimensions of the individual members of the frame

Substituting numerical values,

$$I_{x_o} = \frac{2 \times 20^3 \times 0.193 - 24.43 \times 8.19^2}{\text{minimum } I_{1-2}} = \frac{1449}{\text{minimum } I_{1-2}}$$

Applying Corollaries 1 and 4, the moments of inertia of the elastic weight of the frame about the Y_o-axis passing through the elastic center of the frame may be expressed by the following equation:

$$I_{y_o} = \frac{2h\Theta_{12}}{\text{minimum } I_{1-2}} \cdot \left(\frac{L}{2}\right)^2 + \frac{\phi L^3 (\alpha_{23} - \beta_{23})}{2(\text{minimum } I_{1-2})} \quad \ldots\ldots\ldots\ldots (30)$$

Substituting numerical values,

$$I_{y_o} = \frac{40^2 [20 \times 0.3756 + 0.297 \times 40(0.2437 - 0.1525)]}{2 \text{ (minimum } I_{1-2})} = \frac{6877}{\text{minimum } I_{1-2}}$$

Selecting a basic statically determinate structure as shown schematically in Fig. 5, and realizing that the vertical reaction at the left support is 3.33 kips, the static bending moment diagram of the frame becomes identical to the simple beam bending moment diagram of member 2-3. The basic frame with the static bending moment diagram is shown in Fig. 6.

Placing the origin of coordinates at the elastic center of the frame and denoting positive directions of X_o- and Y_o-axes and the statically indeterminate quantities H, V, and Z as

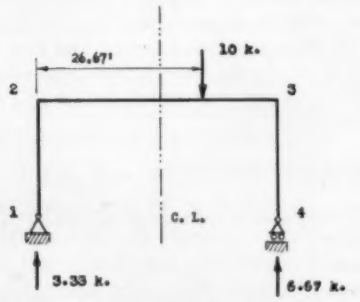

Fig. 5—Statically determinate frame

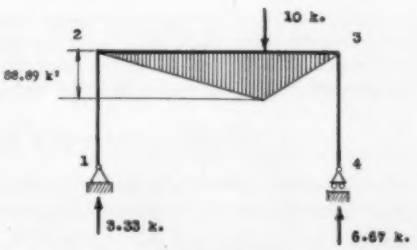

Fig. 6—Diagram of the static bending moment

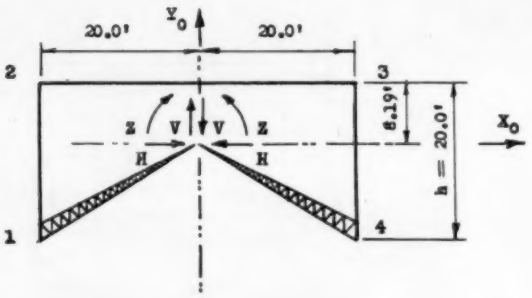

Fig. 7—Application of the elastic center method

shown in Fig. 7, the moment at any section of the frame may be expressed by the following equation:

$$M = M_o + Z + Vx - Hy \quad \ldots\ldots (31)$$

where

$$Z = -\frac{\dfrac{\text{area of modified } M\text{-diagram}}{\text{minimum } I_{2-3}}}{\dfrac{B}{\text{minimum } I_{1-2}}} \quad \ldots\ldots (32)$$

$$H = \frac{\dfrac{\text{area of modified } M\text{-diagram} \times n_o}{\text{minimum } I_{2-3}}}{I_{x_o}} \quad \ldots\ldots (33)$$

$$V = -\frac{\dfrac{\text{area of modified } M\text{-diagram} \times u}{\text{minimum } I_{2-3}}}{I_{y_o}} \quad \ldots\ldots (34)$$

where the modified M-diagram is the reduced static bending moment diagram, whose ordinates are the ordinates of the bending moment diagram multiplied by the factor I_o/I.

M_o = static moment at an arbitrary section
n_o, u = respective ordinate and abscissa of the center of gravity of the area of the modified moment diagram in X_o, Y_o coordinate system
x, y = coordinates of a section of the frame

The area of the modified moment diagram may be determined by means of load constants, R_{23} and R_{32}. Since these constants are the reactions of the modified moment diagram taken as a load, the area of the modified moment diagram, for the member of unit length and $P = 1$, may be expressed as $R_{23} + R_{32}$. In order however, to obtain the absolute numerical value of the area for the girder of length L, and for the given concentrated load P, the factor PL^2 must be used. Thus, the area of modified M-diagram is

$$PL^2 (R_{23} + R_{32}) = 10 \times 40^2 (0.0464 + 0.0566) = 1648$$

The expression, "area of modified M-diagram $\times$ u," may also be formulated in terms of the load constants R_{23} and R_{32}, in this case, by the expression:

$$\frac{PL^2 (R_{32} - R_{23})}{2} = \frac{10 \times 40^2 (0.0566 - 0.0464)}{2} = 3264 \quad \ldots\ldots (35)$$

Substituting numerical values in Eq. (32), (33), and (34) and expressing the reference moment of inertia of the member 2-3 in terms of reference moment of inertia of member 1-2, the statically indeterminate quantities Z, H, and V may be evaluated:

$$Z = -\frac{1648 \times 0.297}{24.43} = -20.04 \text{ ft-kips}$$

$$H = \frac{1648 \times 0.297 \times 8.19}{1449} = 2.77 \text{ kips}$$

and

$$V = -\frac{3264 \times 0.297}{6877} = -0.141 \text{ kips}$$

The final steps of the analysis are the determination of the moments. Employing Eq. (31), the moments at the joints are

$$M_2 = 0 + (-20.04) + (-0.141)(-20) - 2.77 \times 8.19 = -39.91 \text{ ft-kips}$$

$$M_3 = 0 + (-20.04) + (-0.141)(20) - 2.77 \times 8.19 = -45.54 \text{ ft-kips}$$

$$M_1 = 0 + (-20.04) + (-0.141)(-20) - 2.77(-11.81) = 15.49 \text{ ft-kips}$$

$$M_4 = 0 + (-20.04) + (-0.141)(20) - 2.77(-11.81) = 9.86 \text{ ft-kips}$$

GENERAL CONSIDERATIONS IN DERIVING ELASTIC PARAMETERS OF CURVED MEMBERS

The analysis of arches or arched frames, with fixed or hinged supports, and members of variable cross section, also may be conveniently accomplished through the use of elastic parameters. In order to take full advantage of the brevity and expediency of the method, one must have available numerical values of elastic parameters and load constants for various shapes of arches. As will be amplified later, these data are not yet available in the technical literature.

Generally, for any one-dimensional (straight) member, all elastic properties may be completely expressed in terms of its three elastic parameters. For a symmetrical member of the same group, two parameters suffice for evaluation of the member's properties.

In the case of a two-dimensional (curved) member, the number of elastic characteristics is twice that of the one-dimensional member. For example, the moment of inertia of elastic weight is necessarily referred to both axes of the Cartesian coordinate system. Similarly, the location of the elastic center and many other properties of the member are completely defined only if two coordinates are known. Consequently, to express various properties

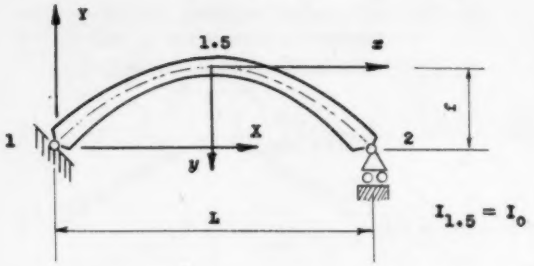

Fig. 8—Coordinate systems used in derivation of elastic parameters and load constants for curved members

of an unsymmetrical two-dimensional member in the directions of two coordinate axes, six parameters are needed.

Taking advantage, however, of the conclusion drawn from Corollary 1, it is seen that the determination of only five elastic parameters is required for the complete description of a two-dimensional member in any one system of coordinates. Three elastic parameters are defined by Eq. (6a); for the other two, the writer introduces the following mathematical definition:

$$\gamma_{12} = \frac{1}{Kf^2} \int_0^K \frac{I_o}{I} \frac{y^2\, dx}{\cos \varphi} \qquad (36)$$

$$\delta_{12} = \frac{1}{Kf^2} \int_0^K \frac{I_o}{I} \frac{y\,(f - y)\, dx}{\cos \varphi} \qquad (37)$$

In the special case of the analysis of a *symmetrical arched member*, the determination of four parameters α_{12}, β_{12}, γ_{12}, and δ_{12} is sufficient.

While the basic expressions for the elastic parameters of a curved member are clearly defined above, some difficulty arises in the actual process of calculation of parameters even for the symmetrical haunched arched member with supports on the same level. For this arch, the mathematical expression of the rate of change of cross sectional area is generally referred to the x-y Cartesian coordinate system with its origin at the crown as shown in Fig. 8, rather than the X-Y system of coordinates in which the elastic parameters have been previously defined by Eq. (6a), (36), and (37).

The system of coordinates with origin at the crown will be retained for the convenience of integration, and consequently the equations of parameters should be redefined. Realizing that the expressions for the elastic parameters of the member are derived from the angles of rotation of its ends when a unit moment is applied to one end, then the elastic parameters α_{12} and β_{12} of the symmetrical arch may be expressed in the x-y coordinate system of Fig. 8 by the following equations:

$$\alpha_{12} = \alpha_{21} = \frac{1}{2L} \int_0^{L/2} \frac{I_o}{I}\, ds + \frac{2}{L^3} \int_0^{L/2} \frac{I_o}{I}\, x^2\, ds \qquad (38)$$

$$\beta_{12} = \beta_{21} = \frac{1}{2L}\int_0^{L/2} \frac{I_o}{I}\, ds - \frac{2}{L^3}\int_0^{L/2} \frac{I_o}{I}\, x^2\, ds \quad \ldots\ldots (39)$$

The elastic parameters γ_{12} and δ_{12} are derived from the condition of a unit moment applied at the crown of the arch. For the symmetrical arch with supports on the same level, oriented in the x-y system of coordinates as shown in Fig. 8, these parameters may be expressed by Eq. (40) and (41).

$$\gamma_{12} = \frac{2}{Lf^2}\int_0^{L/2} \frac{I_o}{I}\,(f - y)^2\, ds \quad \ldots\ldots (40)$$

$$\delta_{12} = \frac{2}{Lf^2}\int_0^{L/2} \frac{I_o}{I}\, y\,(f - y)\, ds \quad \ldots\ldots (41)$$

Replacing ds by the expression defining the rate of change of cross sectional area, integration of Eq. (38) through (41) may be accomplished for a specific arch.

GENERAL CONSIDERATIONS IN DERIVING LOAD CONSTANTS OF CURVED MEMBERS

The determination of the load constants of curved members basically follows the same procedure as described in the previous article for the elastic parameters. Generally, for a two-dimensional or curved member, three load constants are necessary: two constants T and U, to express the load characteristics in the direction of X-axis, and the third constant S, to express the same characteristics in the direction of the Y-axis.

Since the coordinates of a curved member are related to each other by a quadratic expression, each curved member has two independent sets of load constants. One set applies to the vertical loadings on the member, the other to the horizontal loadings.

The load constants of an arbitrary vertical load applied to a symmetrical arch oriented in the X-Y coordinate system, as shown in Fig. 8, are defined by the writer, with respect to the X-axis by the following equations:

$$S = \frac{1}{(WL)Lf}\int_0^{L} \frac{I_o}{I}\,\frac{My\, dx}{\cos\varphi} \quad \ldots\ldots (42)$$

$$T = \frac{1}{(WL)L^2}\int_0^{L} \frac{I_o}{I}\,\frac{M(L - x)\, dx}{\cos\varphi} \quad \ldots\ldots (43)$$

$$U = \frac{1}{(WL)L^2}\int_0^{L} \frac{I_o}{I}\,\frac{Mx\, dx}{\cos\varphi} \quad \ldots\ldots (44)$$

Similarly, the load constants of an arbitrary horizontal load, are defined as follows:

$$S = \frac{1}{(Wf)\,Lf} \int_0^L \frac{I_o}{I} \frac{My\,dx}{\cos \varphi} \qquad (45)$$

$$T = \frac{1}{(Wf)\,L^2} \int_0^L \frac{I_o}{I} \frac{M(L-x)\,dx}{\cos \varphi} \qquad (46)$$

$$U = \frac{1}{(Wf)\,L^2} \int_0^L \frac{I_o}{I} \frac{Mx\,dx}{\cos \varphi} \qquad (47)$$

The integration of Eq. (42) through (47) may be accomplished for a particular curved member if the above equations are redrafted for use in the x-y coordinate system and an appropriate expression defining the rate of change of arch cross sectional area is substituted for $dx/I \cos \varphi$.

NUMERICAL VALUES OF THE ELASTIC PARAMETERS AND LOAD CONSTANTS OF CURVED MEMBERS

With the defined elastic parameters and load constants of the curved member, and with relations formulated by the concept of elastic parameters, the elastic properties of an arch may be readily computed. As a result, an analysis of a complete structure may be accomplished by straightforward, short calculations employing the elastic center method and treating the arch body as a single element with defined properties. The advantage over the tedious method now generally employed is immediately realized.

The problem therefore narrows to determination of the numerical values of elastic parameters and load constants, and their presentation in tabular form for many arched members of constant and variable moment of inertia. The writer has derived numerical values of elastic parameters and load constants for various haunched and tapered arches; these and other data are compiled in book form.*

APPLICATION TO THE ANALYSIS OF ARCHES

For the purpose of illustrating the analysis of a curved structure, the following example is presented. Numerical values of elastic parameters and load constants are taken from the writer's development.†

Example

Find the statically indeterminate thrust of a symmetrical arched bridge with parabolic, tapered, two-hinged arch when loaded by five equal concentrated loads as shown in Fig. 9.

*Leontovich, Valerian, *Frames and Arches*, McGraw-Hill Book Co., New York (in printing).

†Since the author uses a different basic system in derivation of his elastic constants from that employed by R. Guldan, the author's constants in this paper have been converted to match the system used by R. Guldan for straight members. This makes numerical values of the elastic constants of the curved members consistent with numerical values of elastic constants of the straight members used in the previous example.

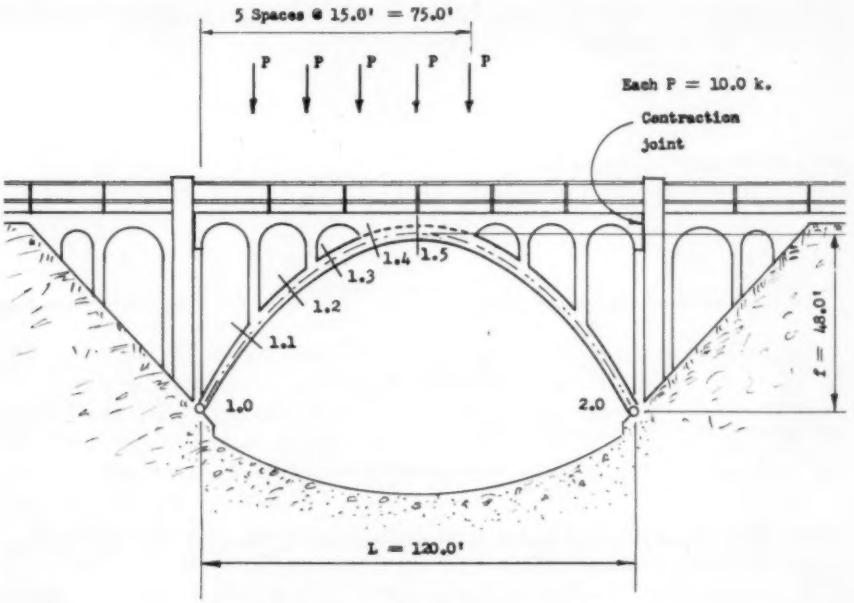

Fig. 9—Two-hinged arched bridge

In this illustrative example only live loads on the arch are considered for brevity of the analysis, with the solution based on the assumption that the system of loads is carried by the columns to the arch and distributed uniformly across its width. The statically indeterminate thrust is calculated in kips per entire arch width. The span and rise of the arch are 120 ft and 48 ft, respectively. The moment of inertia of the arch cross section varies in accordance with the following data:

Section	Horizontal distance	I
1.0		1.52 ft⁴
	0.1 L	
1.1		1.84 ft⁴
	0.1 L	
1.2		2.28 ft⁴
	0.1 L	
1.3		2.88 ft⁴
	0.1 L	
1.4		3.60 ft⁴
	0.1 L	
1.5		4.00 ft⁴

For the analysis of the two-hinged symmetrical arch, only one parameter γ and one load constant S_n of each concentrated load are needed. For the given arch the elastic parameter γ is obtained as a tabular value, 0.8383, and the five load constants corresponding to the five concentrated loads are 0.070, 0.119, 0.146, 0.154, and 0.146 The aggregated value of load constants defined by the symbol S is therefore 0.635.

The moment of inertia of the elastic weight of the arch about the X_o-axis, passing through the hinges may be expressed by the following equation:*

$$I_{z_o} = \frac{\gamma L f^2}{I_{crown}} \quad \ldots\ldots (48)$$

and introducing numerical values,

$$I_{z_o} = \frac{0.8383 \times 120 \times 48^2}{I_{crown}} = \frac{231{,}770}{I_{crown}}$$

The moment about the X_o-axis of the $\frac{\text{area of modified } M\text{-diagram}}{I_{crown}}$ may be expressed as

$$M_{z_o} = PS \frac{L^2 f}{I_{crown}} \quad \ldots\ldots (49)$$

and upon substitution of numerical values,

$$M_{z_o} = \frac{10 \times 0.635 \times 120^2 \times 48}{I_{crown}} = \frac{4{,}389{,}100}{I_{crown}}$$

The statically indeterminate thrust H of the arch may be obtained from the equation:

$$H = \frac{M_{z_o}}{I_{z_o}} \quad \ldots\ldots (50)$$

Substituting numerical values and performing cancellations, $H = 18.94$ kips.

Having calculated the statically indeterminate quantity, the moment at any section of the arch may be obtained from the familiar equation:

$$M = M_o - Hy$$

OTHER APPLICATIONS OF ELASTIC PARAMETERS

Among other significant applications of the elastic parameters, the citing of a generalized definition of flexural rigidity, based on the condition of pure bending, is imperative.† The importance and significance of this generalized definition lies in the fact that it permits the extension of certain concepts to straight and curved members of variable cross section.

To arrive at the expression for flexural rigidity, based on the condition of pure bending, the stored energy of deformation in the member is considered as a criterion. Applying two opposite unit moments‡ to the ends of an arbitrary member 1-2 of Fig. 1, the stored energy of flexural deformation U expressed in terms of applied moments and angular deformations, is

$$U = \frac{M_1 (\alpha^0_{12} + \beta^0_{21})}{2} + \frac{M_2 (\alpha^0_{21} + \beta^0_{12})}{2} \quad \ldots\ldots (51)$$

*E is omitted by the previously stated reasons.

†This definition should not be confused with flexural rigidity of the member as defined through the use of vertical displacement of one support for members with fully fixed ends. For definition of flexural rigidity, based on condition of pure bending, see Timoshenko, S. P., *Strength of Materials*, D. Van Nostrand Co., New York, 1940, Part I, Article 23, p. 88.

‡The in.-lb dimensional system is employed.

Observing that the applied moments M_1 and M_2 are equal to 1 lb-in., Eq. (51) may be written in terms of elastic parameters as follows:

$$U = (\alpha_{12} + \alpha_{21} + 2\beta_{12}) \frac{K}{2EI_o} (1 \text{ lb-in.})^2 \ldots\ldots\ldots\ldots\ldots\ldots (52)$$

With the use of Eq. (7), the stored energy of deformation U may be expressed as

$$U = \Theta_{12} \frac{K}{2EI_o} (1 \text{ lb-in.})^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (53)$$

The term EI_o/Θ_{12} in Eq. (53) defines the rigidity of a member of unit length, based on the condition of pure bending. If the quantity EI_o is the same for members of unit length but of different shape and cross section, a commensurable basis is established for their comparison. The term Θ_{12} is then the criterion of rigidity of a member and therefore is called the *elastic quantum of the member.*

The significance of the elastic quantum is manifold. First, as mentioned, dissimilar nonprismatic members may be logically classified and compared in terms of their elastic rigidity. Secondly, many complex problems involving elastic stability and deflection of straight and curved members of variable cross section may be more readily solved through application of the elastic quantum.

The term rigidity, based on the condition of pure bending, is currently applicable to prismatic members only.* The concept of elastic quantum, however, permits a more generalized interpretation of the meaning of rigidity of a member, so that the definition may be extended to straight and curved members of variable cross section.

For a straight member of constant cross section and unit length, the elastic quantum is unity; the expression for rigidity then reduces to the familiar EI_o. For a haunched member of unit length, and I_o corresponding to the I of the minimum section, the value of the quantum is necessarily always less than unity. It is apparent, of course, that the haunched member is storing less energy of deformation than a corresponding prismatic member and is therefore more rigid.

Tapered members of the same length and E and with I_o corresponding to the moment of inertia of the member's maximum cross section may also be fitted easily into this scheme of classification. Their quanta values are always greater than unity because each tapered member stores more energy of deformation than the prismatic member of corresponding length and EI_o.

The elastic parameters, quanta, and load constants discussed above are dimensionless quantities; however, having been obtained by direct reduction from certain physical properties of the member, they retain very important

*See any engineering mechanics text.

functional relations. As a result, the principle of superposition may be advantageously applied to obtain the elastic parameters, quanta, and load constants of members not presently covered by the tables and charts.

The diverse applications of elastic parameters cannot all be covered in this paper. These other applications not included in the paper are of no less value than those herein presented.

CONCLUSION

Nine corollaries to the moment-area theorems have been formulated, introducing the concept of the elastic parameters and, therein, a new interpretation of the theorems. The classical elastic-center method has been extended through the concept; the usefulness and superiority has been demonstrated by practical examples.

It has also been shown that the extended method possesses the unique advantage of considering a structure of straight or curved members as a single integral unit with precisely defined physical properties. As a result of this approach, complex problems may be readily solved through the use of conveniently tabulated values of elastic parameters and load constants; the tedious semigraphical integration involved in the present method is completely avoided.

The brevity of the presented method has also been shown, inferring particular utility for the analysis of frames suffering "body swing" under load.

Although space limitations preclude further exploration of the concept of elastic parameters, its utility is evident in application to the analysis of space rigid frames, frames on elastic foundations, cases of elastic stability, and other problems of structural engineering.

Discussion of this paper should reach ACI headquarters in triplicate by Aug. 1, 1958, for publication in the Part 2, December 1958 JOURNAL.

CONCRETE BRIEFS

Notes from Field and Office

Topping Pavements with Calcium Aluminate Cement Concrete 1009
Some Effects of Carbon Dioxide on Mortars and Concrete—Discussion 1012
Problems and Practices 1012

Topping Pavements with Calcium Aluminate Cement Concrete

By W. C. HANSEN* and W. W. BRANDVOLD†

INTRODUCTION

Experiments were started in 1951 to study methods of placing a topping of calcium aluminate cement‡ concrete on a portland cement concrete base to provide a surface for airfield pavements that would be resistant to the heat of jet and rocket engines. This is a report on the conditions of slabs and the strengths of beams at the end of 5½ years exposure in the field.

SPECIMENS AND TEST OUTLINE

The specimens for each condition in this study were as follows: one 36 x 36 x 9-in. slab, and four 6 x 6 x 30-in. beams.

The principal purpose of this investigation was to study methods of bonding a calcium aluminate cement concrete topping to a portland cement concrete base. The variables studied were as follows: Type I and Type IA cements were used in the concrete base; and the topping was placed on the base as follows:

(1) within 10 min after the base course was placed;

(2) immediately after the base course had taken its initial set; and

(3) when the base course was 3 days old or more.

Traprock from Dresser Junction, Wisc., and blast-furnace slag aggregate from East Chicago, Ind., were used in the toppings. The temperature of the Lumnite concrete was varied between 60 and 90 F.

MIXING, PLACING, AND TESTING

The aggregates stockpiled in the field were sprayed with water to maintain them in a saturated surface-wet condition. The free water in the aggregate was determined immediately before batching of the concrete. The concrete was mixed for 75 sec in a 6 cu ft non-tilting revolving drum-type mixer.

The portland cement concrete, placed in one layer to a depth of 6 in. in a wooden slab mold, was spaded along the edges to obtain a well-filled mold and then was struck off with a 4-in. channel iron screed in three successive passes, in alternating directions. This was the only finishing of the surfaces of the slabs which were topped within 10 min after placing. The other slabs were given a scratched surface with a steel rake similar to a garden rake. Slabs that were not topped for 3 or more days were cured under wet cloth for 24 hr.

At the time of placing the base course slab, a 3-in. base course was placed in four 6 x 6 x 30-in. steel molds. Excess concrete was put in the molds and was removed with a wooden screed prepared for this purpose, which maintained the thickness of the concrete at 3 in.

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*Member American Concrete Institute, Director of Research, Research Laboratories, Universal Atlas Cement Co., Buffington, Ind.

†Section Leader, Research Laboratories, Universal Atlas Cement Co., Buffington, Ind.

‡The cement used was the product sold under the trade name of Lumnite.

TABLE 1—MIX DATA FOR BASE COURSE

Type of cement	Cement content, sacks per cu yd	*W/C* gal. per sack	Unit weight, lb per cu ft	Air content, percent	Mix proportions by weight	Sand content, percent
IA	6.11	5.7	150.1	4.2	1:1.73:3.88	30.8
I	6.28	5.4	156.0	1.7	1:2.06:3.70	35.7

and screeded the surface. The surface of the base course in each of the four molds was finished in the same manner as the surface of the companion slab. This gave four beams for each condition of placing the toppings. The mix data for the portland cement concrete are shown in Table 1.

After the base course for the slabs was placed, a 3-in. wooden extension was placed on the base course forms. This was filled in one layer with Lumnite cement concrete which was spaded along the edges of the forms and was struck off with the steel screed as described for the base course. This was the only finish given the surfaces of the slabs.

In the case of the base courses, which were 3 days or older before they were topped, a mortar consisting of one part by weight of Lumnite cement and one part of the sand to be used in the topping was brushed thoroughly into the surface of the base course immediately before the topping was placed.

The 3-in. toppings on the 6 x 6 x 30-in. beams were placed in the same manner as that followed with the slabs, except that the surfaces were troweled with a steel trowel to provide a smooth surface for the knife edges of the test machine in the test for flexural strength. The beams were broken on 18-in. spans with the surface as cast facing upward so as to apply the load in the manner in which it would be applied in a pavement. This gave two breaks for each beam. The three sections of the broken beams were broken under compression as modified cubes. This load was applied to the sides of the beam as cast.

Since the strength and ease of placing of Lumnite concrete decrease as the temperature of the concrete increases, toppings were placed with the Lumnite concrete having a temperature of 60 to 70 F in one set of specimens and a temperature of 80 to 90 F in a second set to determine whether or not the

TABLE 2—MIX DATA FOR LUMNITE CONCRETE TOPPING PLACED ON BASE INDICATED

Condition of portland cement base	Cement content, sacks per cu yd	Aggregate	*W/C* gal. per sack	Unit weight, lb per cu ft	Mix proportions by weight	Sand content, percent	Temperature of concrete, F
Type IA concrete base							
Fresh	5.97	Slag	10.7	146.2	1:1.91:3.18	37.5	84
Set	5.45	Slag	13.6	146.2	1:2.31:3.18	42.1	84
Hard	5.51	Slag	12.7	146.2	1:2.31:3.18	42.1	82
Fresh	5.85	Slag	7.6	146.0	1:2.32:3.18	42.2	62
Set	5.85	Slag	7.6	146.0	1:2.23:3.18	42.2	52
Hard	5.68	Slag	7.6	146.0	1:2.73:2.73	50.0	70
Fresh	6.19	Traprock	8.3	147.0	1:1.84:3.24	36.2	63
Set	6.22	Traprock	7.9	147.0	1:1.84:3.24	36.2	60
Hard	6.22	Traprock	7.9	147.0	1:1.84:3.24	36.2	61
Type I concrete base							
Fresh	6.45	Traprock	6.1	162.0	1:2.16:3.52	38.0	88
Set	6.45	Traprock	6.1	162.0	1:2.16:3.52	38.0	91
Hard	6.45	Traprock	6.1	162.0	1:2.16:3.52	38.0	90
Fresh	6.73	Traprock	7.1	157.4	1:1.84:3.24	36.2	60
Set	6.73	Traprock	6.7	157.4	1:1.84:3.24	36.2	62
Hard	6.73	Traprock	6.4	157.4	1:1.84:3.24	36.2	62

TABLE 3—STRENGTH OF 6 x 6 x 30-IN. BEAMS OF LUMNITE CEMENT CONCRETE BONDED TO PORTLAND CEMENT CONCRETE

Aggregate in topping	Type of cement in base	Condition of base	Flexural strength, psi 6 x 6 x 30-in. beams (average of two breaks)				Compressive strength, psi 6-in. modified cubes (average of three cubes)		
			24 hr	1 month	1 year	5.5 year	24 hr	1 month	1 year
Topping placed at 80 to 90 F									
Slag	IA	Fresh	159	595	654	617	1141	1562	1872
		Set	203	610	623	665	1304	2344	2066
		Hard	318	515	613	598	1579	1961	2168
Trap Rock	I	Fresh	308	708	732	779	1558	3115	3319
		Set	325*	819	889	776	1863	4030	2882
		Hard	610*	540*	†	†	2310	2737*	†
Topping placed at 60 to 70 F									
Slag	IA	Fresh	50	561	697	684	705	2744	3306
		Set	76	595	673	713	739	3574	2974
		Hard	362	511	603	605	1829	2622	3519
Trap Rock	IA	Fresh	71	603	671*	768	527	3155	3226
		Set	71	629	751	903	770	4148	4905
		Hard	535	531	753	753	2126	3085	4275
Trap Rock	I	Fresh	149	662	779	733	1128	4209	5474
		Set	187	708	871	850	1700	5298	5001
		Hard	462	625	661	680	2164	3925	5017

*One break.
†Portland cement base cracked.

temperature of the mix at time of placing would appreciably affect the bond between the base and topping courses. The higher temperatures were obtained by placing the required amount of mixing water heated to about 160 F in the mixer and rotating the mixer about 1 min before the cement and aggregate were added. The mix data for concrete topping are given in Table 2.

The toppings on the slabs and beams were sprayed with a membrane-forming curing compound as soon as possible after placing. The molds were removed at the age of 24 hr and soil was placed around the slabs so that the surfaces were flush with the surfaces of the ground. The beams were similarly embedded in the soil. The beams were broken in flexure at the ages of 24 hr, 1 month, 1 year, and 5½ years, and as modified cubes at ages of 24 hr, 1 month and 1 year. The results of these tests are given in Table 3. The sections of the broken beams at the 5½-year period were used in attempts to break the bond between the two types of concrete. This was done by the driving of a steel wedge into the specimen at the interface of the topping and base course. There was no evidence of poor bond on any of the beams; in no case did the specimen break at this interface.

The only study of the slabs has been to tap the surfaces in an effort to detect any breaking of the bond between the two types of concrete and to examine them for cracks. So far, there is no evidence of any failure of this bond.

DISCUSSION

The principal object of this study was to expose slabs of concrete, in which portland cement concrete was topped with a calcium aluminate cement concrete, to normal weather conditions, which include many cycles of freezing and thawing each year, for the purpose of determining whether or not the bond between the two types of concrete would remain intact under these conditions. As far as can be determined, this bond has remained perfect for about 6 years. The slabs are to be left undisturbed and observations will continue.

Table 3 shows that the flexural strengths of the beams equaled or exceeded 600 psi at the ages of 1 and 5½ years. The best strength generally was obtained when the topping was placed just after the concrete of the base course had taken its initial set. However, strength differences were not large.

Some Effects of Carbon Dioxide on Mortars and Concrete—Discussion

By A. STEOPOE*

The writer read with interest the paper by Leber and Blakey, "Some Effects of Carbon Dioxide on Mortars and Concrete" (ACI JOURNAL, Sept. 1956, *Proc.* V. 53, pp. 295-308), since a long while ago he studied the chemical processes which occur when aggressive agents react on the hardened cement.

On p. 304, Leber and Blakey state: "If carbon dioxide decomposes the products of the hydration of di- and tri-calcium silicates, water and hydrous silica will be formed. This hydrous silica is insoluble in diluted hydrochloric acid (Meyers[7]), a fact which could lead to serious mistakes, when determining cement in old and carbonated concrete by the Florentin method."

The work by Meyers was published in *Rock Products* in 1949. The writer observed the same process 14 years earlier and published the results in the German periodical *Zement*, 1935, No. 50, under the title "The Action of Carbon Dioxide on Hardened Cement." In this paper the formation of insoluble silica was demonstrated experimentally.

The mistakes caused by presence of insoluble silica in determining proportions in old and carbonated concrete by the Florentin method were also reported by the writer in "The Determination of the Mix Proportions in Mortar and Concrete" (published in 1934, also in *Zement*).

Following the above studies, the writer modified the method of analysis of concrete, introducing an alkaline treatment after the acid one. Two papers concerning this analysis were published in the German periodical *Tonindustrie Zeitung* (1937, No. 11, and 1938, No. 33).

*Bucharest, Rumania.

Problems and Practices

A series relating to "down-to-earth, everyday" concrete problems which attempts to give brief answers to the more common (and sometimes uncommon, too) questions asked about concrete design and construction practices.

To some, the answers will seem simple and obvious; others may prove controversial.

All ACI members are invited to participate—either by submitting an inquiry, or even better, by telling JOURNAL readers how an intriguing problem was solved. It may well be that readers will be able to suggest more practical solutions than those presented.

Q. How should ordinary gypsum plaster be applied on dense concrete interior walls or ceilings to insure reliable bond?

A. According to the Building Research Station (England) the following are the main points that should be considered:

1. If it is known that concrete surfaces are to be plastered, steps should be taken in consultation with the plastering contractor, to provide some mechanical key on the soffit by using forms with rough surfaces, by suitable retarders, or special inserts in the forms to provide keys.

2. Forms should be level and true to enable a single coat of plaster to be used; joints should be tight, to avoid "fins."

3. Brush surface with a wet brush just before plastering (water should not be visible on the surface) or use a proprietary bonding liquid of the polyvinyl acetate type according to makers' instructions.

4. Use one-coat work if possible; the total thickness of two-coat work should be not more than ⅝ in.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Bending moments in cantilevered roadway slabs (Biegemomente in auskragenden Fahrbahnplatten)

G. Dehnkamp and H. Krosse, *Der Bauingenieur* (Berlin), V. 32, No. 2, Feb. 1957, pp. 52-56
Reviewed by Aron L. Mirsky

Usual method of analysis of curb cantilevers of bridge slabs is to assume them fixed at edge (curb) girder. Authors develop approximation method of analysis of slabs cantilevering on one or both sides and loaded on the cantilevers, with elastic behavior of the slab taken into account. A numerical example of a concrete deck slab 9.5 m out-to-out supported on two girders 4.5 m apart is included. Discussion of paper is printed in July 1957 issue, pp. 283-284.

Construction

Concrete hangars spread folded slab wings

Engineering News-Record, V. 160, No. 8, Feb. 20, 1958, pp. 46-51

Describes five large new cantilever folded plate hangars, including the first, built in 1956 for TWA at Kansas City, and four additional similar hangars presently under construction. Discusses in detail and illustrates different forming methods used.

Composite steel and concrete construction

The Engineer (London), V. 204, No. 5310, Nov. 1, 1957, p. 646
Reviewed by Aron L. Mirsky

Description of a "new method" involving connecting concrete floor slabs, acting as the compression member of the composite section, to steel tension flanges (inverted T's: rolled steel joists cut in half, or built of plates and angles, etc.) by reinforcing-steel spirals welded to top of steel portion and embedded in the concrete.

In view of the fact similar methods have been used in this country ("Alpha") and elsewhere for quite a few years, reviewer found it surprising that "the development of this method of construction has involved a considerable amount of testing . . . ."

Design and construction of the Apapa wharf extension, Lagos

Douglas Cecil Coode, *Proceedings*, Institution of Civil Engineers (London), V. 7 (session 1956-57), July 1957, pp. 499-519 (discussion, pp. 519-536)
Reviewed by Aron L. Mirsky

Of particular interest in this Nigerian project is the wharf wall. The bed of the harbor consists of fine and coarse sand, with some silt and pockets of clay; hence wall was built of concrete blocks set in sloping bond on a rubble base: each "slice" can thus settle independently of adjacent slices. Further, effect of variable loading during construction on settlement and tilt was carefully studied, and backwards lean of the slices chosen so final slopes would not exceed desired amounts.

Uses of lightweight concrete (Contribution a l'étude des problémes posés par l'utilisation des bétons légers)

Henri Bodecher, Supplement to *Annales* de L'Institut Technique du Batiment et des Travaux Publics (Paris), V. 10, No. 120, Dec. 1957, pp. 1321-1332; discussion pp. 1332-1333
Author's Summary

Siporex cellular concrete consisting of Fontainebleau sand, binder, and aluminum powder, and treated in the autoclave in such

A part of copyrighted Journal of the American Concrete Institute, V. 29, No. 11, May 1958, *Proceedings* V. 54. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

a way as to produce a silicocalcareous reaction may vary in density from 0.4 to 0.8, and its strength is such that it may be used for bearing walls on several stories. Presents various examples of the use of cellular concrete, in particular the construction site at Noisy-le-Sec, for which details and costs are given.

Construction Techniques

Composite construction for apartment buildings (Le complexe acier-béton appliqué a la construction du gros oeuvre des immeubles)

JEAN DUTHEIL, Supplement to *Annales* de L'Institut Technique du Batiment et des Travaux Publics (Paris), V. 10, No. 117, Sept. 1957, pp. 830-844; discussion pp. 845-846

AUTHOR'S SUMMARY

Describes the *acier-béton* (steel-concrete) method which combines a steel framework with reinforced concrete.

A quickly erectable light steel framework comprises light section stanchions which have only to support a minimum dead weight during erection. On these stanchions are assembled transverse steel members or prefabricated reinforced concrete members incorporating steel elements in which the connecting parts for welding or riveting are exposed. After assembly, the stanchions and connecting members are embedded in concrete.

Rapid construction by the sliding form method (Les coffrages glissants mode de construction rapide)

EMILE NENNIG, Supplement to *Annales* de L'Institut Technique du Batiment et des Travaux Publics, V. 10, No. 111-112, Mar.-Apr. 1957, pp. 332-375; discussion p. 376

A general discussion of the over-all problems in concrete construction by sliding forms. Emphasizes the necessity for a coordinated operation which can result in extremely rapid construction with a considerable saving in labor. The examples of construction shown and described include silos, cooling towers, reservoirs, chimneys, and multistory buildings. Also deals with the theoretical analysis, design of forms, rate of movement related to quality of the concrete, friction, and jacking forces required. Outlines the elements of design for sliding form systems.

Preparing high strength concrete (La préparation des bétons a hautes résistances)

H. MANCHE, *Revue des Matériaux de Construction* (Paris), No. 498, Mar. 1957, pp. 83-89; No. 505, Oct. 1957, pp. 301-306; No. 506, Nov. 1957, pp. 332-339

Reviewed by PHILLIP L. MELVILLE

A comprehensive review of concrete manufacturing to obtain optimum results. Control of materials is emphasized. Efficiency in operation is discussed with a review of most modern equipment and its effects on quality control and labor costs. In conclusion, it is stressed that modern efficient concrete mixing plants require adequate technical know-how for most advantageous operation.

Design

Cracks in reinforced concrete (Fissuration des corps fragiles et du béton armé)

L. P. BRICE, Supplement to *Annales de L'Institut Technique du Batiment et des Travaux Publics* (Paris), V. 10, No. 109, Jan. 1957, pp. 82-92

AUTHOR'S SUMMARY

Studies the tensile cracks in brittle bodies and in reinforced concrete. Sets up formulas giving the mean spacing of the cracks and their width. With these formulas it is possible to calculate, from characteristics of the section, the maximum stress in the reinforcing steel for a determined mean width of the cracks.

Materials

Study of properties of fly ash cement (Etude des propriétés du ciment aux cendres volantes)

MICHEL VENAUT, *Revue des Matériaux de Construction* (Paris), No. 506, Nov. 1957, pp. 309-317

Reviewed by PHILLIP L. MELVILLE

Two fly ashes used in cement manufacture were studied. A large number of tests on 16 mortar mixes were performed using 4 x 4 x 16-cm bars. Fly ashes were substituted for 25 to 40 percent cement. Pozzolanic reaction started slowly. At 20 C it increases quickly. It increases by grinding and heat. Fly ashes reduce shrinkage as a function of the amount used and of the fineness of the particles. The heat of hydration was only moderately reduced.

New process for electronic microscope study of hardening cement paste (Nouveau procédé pour l'etude au microscope electronique des pâtes de ciment susceptibles de durcissement)

P. BERNARD, *Revue des Matériaux de Construction* (Paris), No. 507, Dec. 1957, pp. 351-361
Reviewed by PHILLIP L. MELVILLE

A study of cement pastes with the electronic microscope under carefully controlled conditions. Results of morphological examination of specimens made with freshly boiled distilled water and commercial grades of portland cements at low water cement ratio are discussed.

Pavements

Pavement condition surveys—Suggested criteria

Special Report No. 30, Highway Research Board, 1957, 61 pp.

A discussion of problems involved in pavement condition surveys, definitions of defects, and suggested forms which have been developed to facilitate recording and studying data obtained.

Prestressed Concrete

Computation of prestressed reinforced concrete cylindrical shells used for containers (in German)

K. BUYER, *Beton und Stahlbetonbau* (Berlin), V. 52, No. 5, May 1957, pp. 104-111
APPLIED MECHANICS REVIEWS Feb. 1958 (Long)

Prestressing is applied to neutralize traction stresses in wet surfaces in water containers (reservoirs) formed by closed cylindrical shells, flat floors, and flat or curved roof. The computation and analysis of stress of some reinforced concrete reservoirs built in Germany are given.

Selection of the allowable initial prestress (Choix des contraintes admissibles initiales dans les aciers pour béton précontraint)

M. A. CHAGNEAU, Supplement to *Annales* de L'Institut Technique du Batiment et des Travaux Publics (Paris), V. 10, No. 120, Dec. 1957, pp. 1353-1358
AUTHOR'S SUMMARY

The falling-off of prestressing in a prestressed concrete construction is due to the slow deformations of the concrete and to its shrinkage, and also to the action of exterior forces.

For a same initial prestressing, it is advisable to use steels under high tension. A certain limit must not be passed, a limit which may be fixed only after an examination of the tensile strength curves of these steels.

Measured losses in prestress (Expériences de relaxation des contraintes dans le béton précontraint)

G. DAWANCE and A. CHAGNEAU, Supplement to *Annales* de L'Institut Technique du Batiment et des Travaux Publics (Paris), V. 10, No. 120, Dec. 1957, pp. 1344-1353
AUTHORS' SUMMARY

Tests begun in 1952 with extruded steel wire have been continued and new tests have been carried out on patented steel wires. The present report gives the results obtained in the two cases for different rates of initial compressions of the concrete. The determination of the stresses of the steels is carried out from the measurement of the frequency of transversal vibration.

Properties of Concrete

Investigation of the effect of some pozzolans on alkali reactions in concrete

A. H. M. ANDREASEN, K. E. HAULUND CHRISTENSEN, and P. BREDSDORFF, *Progress Report* L-1, Committee on Alkali Reactions in Concrete, Danish National Institute of Building Research and the Academy of Technical Sciences, Copenhagen, 1957, 88 pp., 12 D.kr.

Report is concerned with studies made up to 1956 investigating the possibilities of producing from Danish raw materials suitable admixtures to concrete, preferably pozzolans, for the prevention of alkali reactions.

The report is divided into two parts. Part I comprises: (1) survey of the development that the use of pozzolans in the production of mortar according to the literature has undergone in the course of time; it is attempted, on the basis of test results and theories available in the literature, to give a review of the effects of pozzolans both hydraulically and as reducing alkali reactions; (2) brief account of the test procedure and of the results obtained, as well as a discussion of these; (3) detailed description of methods

employed; tables and graphs are presented of individual results. Part II comprises: (1) investigation of the various causes for the dispersion of the results obtained in the main tests and an account of the results obtained for three pozzolans by a modified test procedure with improved reproducibility; (2) two appendices, the first on inaccuracies in weighing as a possible source of experimental error and the second, which consists of statistical analysis of the data presented in Part II.

Neutron moisture-meter for concrete

J. Pawliw and J. W. T. Spinks, *Canadian Journal of Technology* (Ottawa), V. 34, 1957, pp. 503-513; *Building Science Abstracts* (London), V. 30, No. 7, July 1957, p. 204

Highway Research Abstracts Feb. 1958

A portable instrument for determining rapidly the total water content of a plain or lightly reinforced concrete member without destruction is described. The fast neutrons from a Ra-Be source (10 or 50 Me) are converted into slow neutrons by the water molecules and are then counted with a BF_3 tube (2-4 in. long). The meter, which can be handled without exposure to excessive radiations, from the source, is calibrated against concrete cylinders of known water content. The sensitivity is less than for the moisture-meter used in soils, because it is only placed against the surface of the concrete.

Effect of coarse aggregate on the mode of failure of concrete in compression and flexure

R. Jones and M. F. Kaplan, *Magazine of Concrete Research* (London), V. 9, No. 26, Aug. 1957, pp. 89-94

Highway Research Abstracts Feb. 1958

Experiments have been made to determine the flexural and compressive strengths of concretes containing 13 different coarse aggregates. Observations were made by an ultrasonic pulse technique to detect the onset of cracking prior to failure.

The main results were as follows: concretes containing smooth gravels began to crack at lower compressive stresses than did concretes containing coarser-textured aggregates. The compressive strengths did not show such appreciable differences. Concretes which best resisted pre-cracking in compression also gave highest flexural strengths.

Revibration of concrete (Revibrering av betong)

Hans Georgii, Lars Ranhem, and Nils Wretlind, Examensarbete vid Institutionen for Byggnadsteknik, KTH (Stockholm), 1956, 35 pp.

Reviews the research on revibration effects on concrete, including references to United States, British, German, French, and Dutch publications. The experimental data by the authors concern principally the effect of revibration for the improvement of bond strength and compressive strength for concrete with various consistencies. Particular improvement in the bond on upper bars in deep members was noted.

General

Engineering college research review

Engineering College Research Council, American Society for Engineering Education, 1957-59 Edition, 408 pp., $2

An exhaustive analysis and location guide for engineering and associated science research activities and capabilities in 110 colleges and universities throughout the country.

Exploratory fire tests with small-scale specimens

Harry D. Foster, *ASTM Bulletin*, No. 228, Feb. 1958, pp. 66-67

Small-scale fire test specimens are usually used in perfecting new materials and construction details before resorting to the standard fire test for establishing acceptable fire-resistance ratings for use in building regulations. This paper points out certain matters which should be observed in making small-scale tests.

Manual on reinforced concrete (Traité de béton armé III)

A. Guerrin, Editions Dunod, Paris, 315 pp.

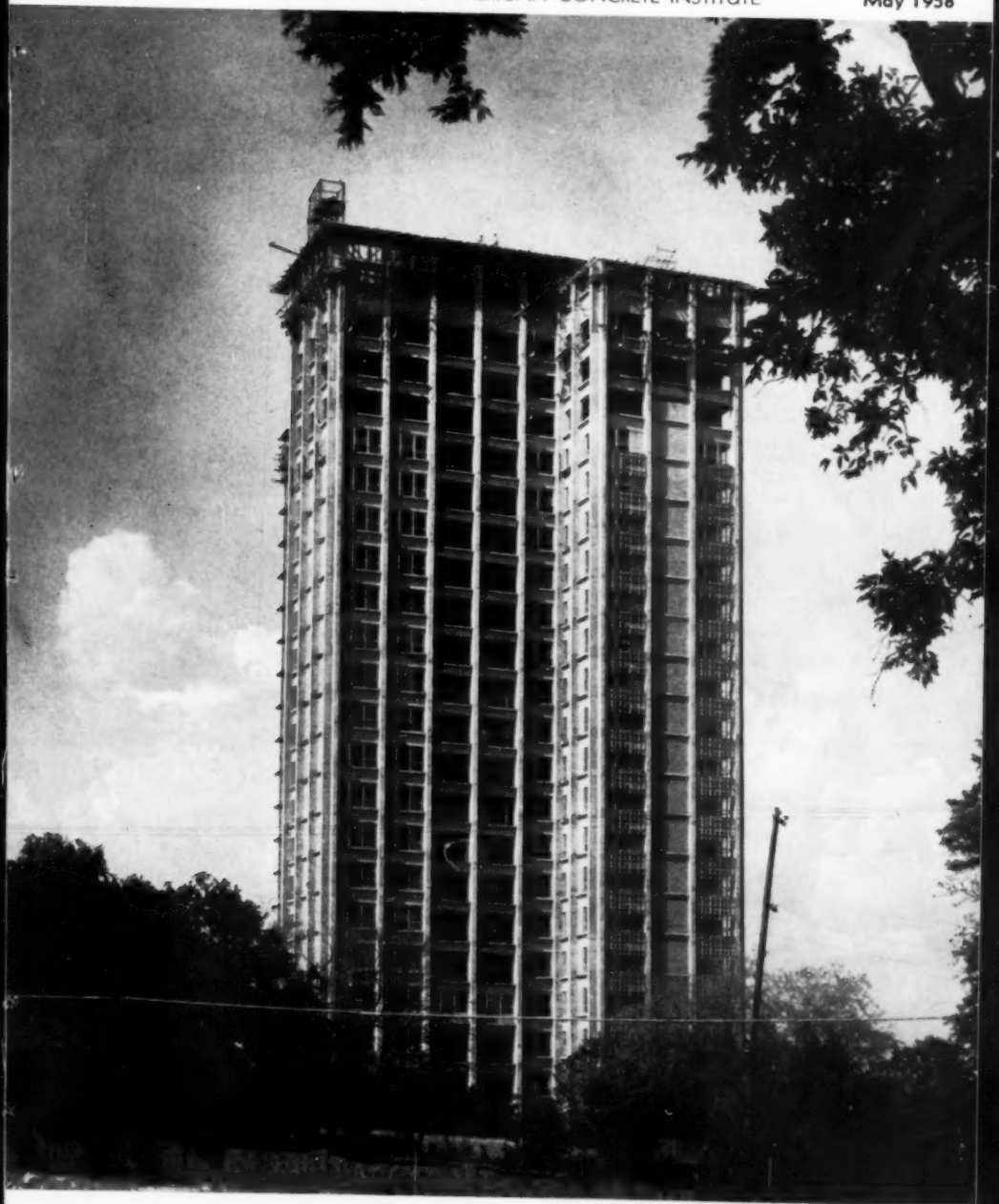
Applied Mechanics Reviews Feb. 1958 (Kezdi)

Book consists of three main chapters: first gives short review of the soil mechanics of foundations; second deals with the dimensioning and design of the structural elements of foundations; third chapter presents some features of special foundation methods.

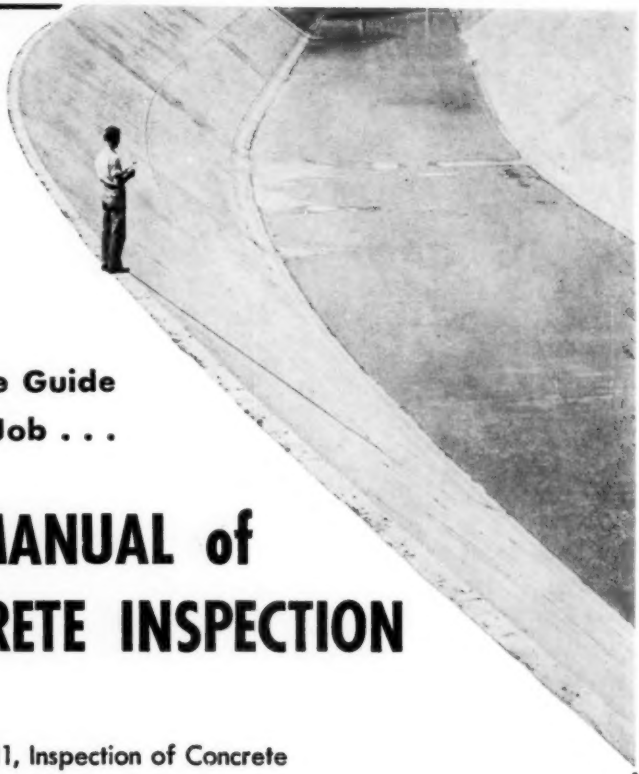
The book is an excellent tool and manual for the practising engineer; many numerical examples and practical hints make it useful, especially for young designers.

ACI News Letter

Vol. 29, No. 11 JOURNAL of the AMERICAN CONCRETE INSTITUTE May 1958



Dallas apartment building . . . p. 3



Pocketsize Guide to a Big Job . . .

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ACI

News Letter Contents

May 1958

On the Cover—This 24-story apartment house in Dallas, Tex., is the subject of Frank W. Chappel's paper on p. 929 of this month's JOURNAL. The completely air-conditioned 102-unit structure makes use of lightweight structural concrete (strength up to 6000 psi) reinforced with alloy steel bars. Precast concrete grills shown installed on part of the frame shield much of the wall and window areas from the sun. For the full story, read "High-Strength Steel and Concrete Result in Minimum Column Sizes."

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Institute best seller now in revised edition

ACI Concrete Primer

MUCH IN DEMAND during the past 30 years, the *Concrete Primer*, by Franklin R. McMillan, consulting engineer, Asheville, N. C., appears this month in new dress—a revised edition brings developments of the past three decades into the question-and-answer format of this long-popular book. Mr. McMillan, then director of research for the Portland Cement Association, first presented the *Concrete Primer* to the Institute at its 24th annual convention Mar. 1, 1928. Later that year it was issued as a special ACI publication; it also appeared in ACI *Proceedings* V. 24 and was awarded the Wason Medal for the "most meritorious paper" that year. Since then the concise pocket book has gone through a dozen printings totaling nearly 85,000 copies printed in English by the Institute. Countless other copies of its text have been reproduced by special permission in Russian, Portuguese, Spanish, Scandinavian, and other languages.

Book size increased

Expanded from 48 to 80 pages, the revised booklet retains the handy $4\frac{3}{4}$ x $7\frac{1}{2}$-in. page size as well as the question-and-answer form of the earlier work. It develops in simple terms the principles governing concrete mixtures, and shows how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete. In the 30 years since the first edition of the *Primer*, great improvements have been made in the machinery and devices for preparing materials; for accurately proportioning the ingredients; for transporting, consolidating, and protecting the concrete. However, with the single exception of the

Continued on p. 4

addition of controlled amounts of entrained air, the *fundamentals* of making durable concrete remain unchanged.

Main changes in the text are thóse which take into account the concept of air entrainment, the recognition of reactive aggregates, the use of several cement types, and high-frequency vibration in placing the fresh concrete. Mr. McMillan has included notable progress in the manufacture of portland cement within the past 30 years, including changes in chemical composition and grinding practices, use of high-early-strength cements; his revised work also cites the five cement types now specified by the American Society for Testing Materials.

The section on the effect of freezing temperatures on concrete strength has been modified, and curing practices which regulate temperature and moisture are given more attention. Reflecting the important influence of technical advances, the section on mixing time has been enlarged to cover transit mixing and new mixer designs.

ORDER BLANK FOR
NEW *Concrete Primer*
ON PAGE 38

The three-page section on air entrainment is entirely new, and air entrainment has been a significant factor in the extensive revisions and enlargement of the section on proportioning. Questions and answers regarding tests and records have increased considerably to include soniscope, rebound, and indentation tests.

Franklin R. McMillan

Honorary member of the Institute, Wason Medalist and long an active ACI worker, Franklin R. McMillan has served in various administrative capacities and on technical committees since joining ACI in 1916. He was president of the Institute in 1936, and has contributed extensively to the literature on concrete. Some 20 of his technical papers and discussions have appeared in the ACI JOURNAL, including the two initial chapters of the "Long-Time Study of Cement Performance in Concrete."

Mr. McMillan's committee activities have been extensive, not only in the American Concrete Institute, but in the American Society of Civil Engineers and the American Society for Testing Materials. An outstanding contribution was his work in connection with the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, beginning with the second Joint Committee in 1921 and continuing this work as secretary of the third Joint Committee which reported in 1940.

A civil engineering graduate of the University of Minnesota in 1905, he engaged in railroad engineering and construction, building construction, and irrigation surveys for about 4 years. Mr. McMillan then became an instructor and later assistant professor at the University of Minnesota where he became intensely interested in concrete and reinforced concrete. He joined the Portland Cement Association in 1924 as an associate engineer. In 1926 he became manager of the structural and technical bureau there, and the following year he became PCA's director of research. After 20 years of noteworthy service in that position, he became assistant to the vice-president in charge of research and development in November, 1947. After a year in that capacity, he retired from the Portland Cement Association, and has since practised privately as a consulting engineer.

Aides in Primer revision

T. C. Powers, Portland Cement Association, Chicago, assisted Mr. McMillan in revising the *Primer*, as did the following members of an ACI committee selected to read the manuscript and aid in its preparation: the late H. S. Meissner, U. S. Bureau of Reclamation; Douglas E. Parsons, National Bureau of Standards, Washington, D. C.; I. L. Tyler, Portland Cement Association, Chicago; Stanton Walker, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D. C.; and C. E. Wuerpel, Marquette Cement Manufacturing Co., Chicago.

Primer for sale now

The *Concrete Primer* is now available to Institute members at 50 cents per copy; price to nonmembers is $1. An order blank appears on p. 38 of the News Letter.

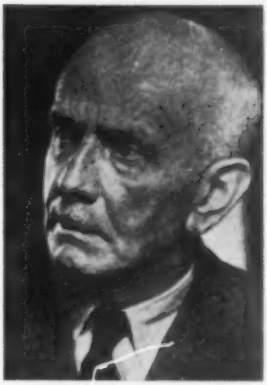
From the notebook of an ACI member, a historical glance at some . . . .

Pioneers in Prestressed Concrete

By J. J. Polivka*

The World Conference on Prestressed Concrete held last summer in San Francisco, with an attendance of some 1200 participants from 43 nations, dramatized the enormous development that prestressed concrete has undergone as a structural material.

Prestressed concrete principles and advantages have been widely discussed and are already familiar. It seems appropriate to pause briefly in this advance and recognize some of the engineers and inventors responsible for this concrete progress.



Karl Bernhard Wettstein in 1919 developed the idea of prestressing concrete for mass-produced structural members, received his first patent in 1921

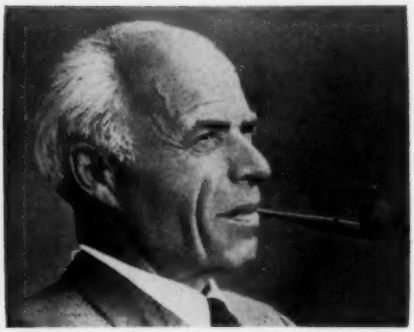
Gustave Magnel, 1889-1956, was professor at the University of Ghent, Belgium, developed systems of prestressing in cooperation with the Blaton contracting firm of Brussels

Jackson devises post-tensioning scheme

Peter H. Jackson, a San Francisco engineer, was definitely at the cradle of prestressed concrete. His patent entitled "Construction of Artificial-Stone or Concrete Pavements" was filed in 1886 and granted in 1888. Essentially Jackson's details and methods conformed with modern post-tensioning. Referring to prestressing reinforcement as "metallic ties," his specifications stated: "When the metallic ties are cemented to the enclosing material along their length, they are incorporated with the mass as long as they hold to it, and any pulling force applied at the end of the tie and compressively resisted by the abutting end of the beam, which is the fulcrum, is not tensilely felt by the tie in the enclosing material while held to it. In order to produce tensile strain on the tie at a suitable distance from the ends, I use tubes or sleeves that will fit closely on the tie with the other end against the abutting plate or side of girder, the enclosing material at the end being cemented to the sleeves and the tie being free to slide inside. After the concrete of the beam has become hard and strong and the tie tensilely-strained inside of the sleeves, that part of the bottom of the beam is compressed and the beam strengthened. These sleeves or coverings of the tie may be of any shape to conform to the shape of the tie, and are preferred to be of metal, or may be any other material that will separate the tie from the material of the beam and permit it to slip through it, such as thick cloth, paper, blacklead, clay, etc."

During the following 10-year period, several improvements and changes in prestressing methods were patented by Doering, Mahal, Koenen, Lund, and others. Of

*Member American Concrete Institute, Consulting Engineer, Berkeley, Calif.



Franz Dischinger, 1888-1953, German educator and engineering consultant, received several patents. In 1934 he proposed a system using post-tensioning ties, hanging in curved lines and externally engaging the reinforced concrete elements, which may be divided into several sections at the joints. Prestressing is by the dead weight of the structure only, without assuring freedom from cracks; or by tensioning to such an extent that freedom from cracking can be guaranteed

particular interest in the United States was Charles R. Steiner's 1908 patent, which stated: "In the various systems of reinforced concrete construction previously in use the metallic tension rods or bars have always been embedded in the concrete without imparting any strain during the setting and hardening of the concrete. In the new system here proposed, however, initial strain is imparted to the tension rods, while the concrete is gaining the coherence and hardness within the next few hours, after being placed. Since there is no appreciable bond between the concrete and the rods at the start of this operation, while at the outside of the concrete resistance increases, there is provided an anchorage at ends."

Wettstein active in Europe

Unfortunately these attempts to make prestressed concrete applicable to larger scale, practical construction had little success. To quote Curzon Dobell who traces the history of patents on prestressed concrete, " . . . we must jump 20 years from 1908 to 1928,* during which time, with one exception, we have no record of any technical writings or patents on the subject. This exception is K. Wettstein . . . who as early as 1919 produced . . . prestressed concrete planks reinforced with highly tensioned wire . . ." Unfortunately Wettstein's methods are mentioned in several sources as the inventions of a German, Ewald Hoyer, who applied Wettstein's ideas and built huge factories in Germany under Nazi's support.

Moved by the critical lumber shortage in some European countries after World War I, Wettstein, then a general contractor in Most, Bohemia, conceived the idea of replacing the most critical boards and planks down to 1 in. thickness with concrete slabs from 0.4 to 1.4 in. thick. In 1919, he began production of prestressed cross-reinforced slabs with two-way strength hitherto attained only in plywood and steel sheets. High strength concrete made with special cements, admixtures, and appropriate fine aggregates of good quality was used. Mechanical vibrators were installed in the first factory in Most, and curing was done in a four-track tunnel. Large sheets were put in steel crates and submerged in hot water tanks. The efficiency of high temperature curing was later also used by Eugene Freyssinet, one of the most original and scientific pioneers in prestressed concrete.

The importance of very thin wires was emphasized in Wettstein's 1921 patent, and in 1923 his commercial literature stated, "Bond is increased approximately 100 percent if a wire of certain diameter is replaced by four wires of half thickness." In the early stages, Wettstein was using special mechanical

*In *The Principles of Prestressed Concrete* by P. W. Abeles (Frederick Ungar Publishing Co., New York, 1949), the work of R. H. Dill, Alexandria, Neb., is cited. He ". . . appears to have been the first, in 1923-25, to propose that this should be carried out by 'post-tensioning.'

"In Dill's system, bonding between the concrete and its reinforcement is prevented by coating the latter with a plastic substance, and the stretching process is carried out after the greater part of the shrinkage has taken place, thus largely avoiding the loss of prestress which rendered the earlier proposals ineffective . . ."

The reader who wishes to explore further will want to refer to Dobell's paper, "Patents and Codes Relating to Prestressed Concrete," ACI JOURNAL, May 1950, *Proc.* V. 46, pp. 713-724. Mr. Dobell has commented on the work of Dill. Mr. Polivka's discussion of the Dobell paper, ACI JOURNAL, Part 2, Dec. 1950, *Proc.* V. 46, pp. 724-1-724-5, provides more detail on the work of Wettstein.—EDITOR

Paul W. Abeles, London, active as a consulting engineer today, has made numerous contributions to prestressed concrete theory, particularly in the field of "partial" prestressing

Mr. Polivka stands on a 12 in. wide, 3/8 in. thick, prestressed concrete plank of 4-ft span—no noticeable deflection. The board, shipped transcontinentally without wrapping or protection against damage, arrived in perfect condition

equipment for prestressing 0.05-in. wires with ultimate strength of 370,000 psi. Later, wires of lower strength—200,000 to 285,000 psi—proved more economical. Numerous tests were made of Wettstein's product in Europe, including load tests by the author at the Technical University of Prague in 1924. Slabs 20 x 80 in., with the roof purlins spaced at 40 in., showed their first fissures under a live load of 88 psf, and failure occurred under 165-psf load.

Slabs with insulating cores

In 1925 Wettstein began manufacture of prestressed slabs with insulating cores of lightweight foam, pumice, or slag concrete. Both the upper and lower layers of the slab were prestressed in two directions. In 1934 the author participated in tests of panels of this type, 6.5 x 1.6 ft, with a 1.2 in. thick insulating layer, and top and bottom prestressed sheets only 0.3 in. thick. Forty-eight high strength wires of 0.035-in. diameter were used longitudinally in upper and lower sheets; 25 were used transversely in both sheets. The first fissure appeared in these panels at a uniformly distributed live load of 86 psf, and failure was at 230 psf.

Long-term performance

In one large factory in Duchcov, Bohemia, built in 1927, roof plans were tested after 17 years, results indicating that the original 230,000 psi prestress had been practically maintained. Similar good performance has been reported in other locations. The roof of Wettstein's original factory in Most was dismantled in 1942 after 20 years of service. These boards showed not the slightest cracks, and no rusting was noticeable on the extracted wires.

Partial prestressing

Partial prestressing was invented by the late Fritz Emperger, internationally known consultant and former professor at the Technical University of Vienna, who was elected to honorary membership in the American Concrete Institute in 1932. Certain improvements in partial prestressing were proposed by Paul W. Abeles, London. Dr. Abeles, Eugene Freyssinet, and the late Gustave Magnel are the three authors who have contributed to basic developments and

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applications of prestressed concrete with the largest number of publications in several languages, on the basis of records in ACI's "Prestressed Concrete Bibliography."

Magnel was formerly professor at the University of Ghent, Belgium, noted as consulting engineer of the Walnut Lane Bridge in Philadelphia, the first prestressed concrete bridge built in the United States in 1949.

Eugene Freyssinet, eminent structural engineer, conceived some of the basic principles of prestressed concrete more than a half century ago, as he remarked in a letter to the writer in 1951: "I would like to state—as I used to say in the past when much less attention was given to my ideas than today—that the essential characteristics of prestressed concrete to which its remarkable properties are due do not depend on prestressing of steel reinforcements, neither on previous compression of concrete in the tension zones, because both conditions can exist, at least to a certain extent, without attaining the essential properties of prestressed concrete. These properties are uniquely due to an exactly determined intensity of compression in the concrete, to the condition of inequality which lets re-enter all possible strains within the elastic margin. These are essential characteristics which I have expressed in 1928 after my patient efforts of emphasizing an idea conceived already in 1903, an idea that I have investigated experimentally with a tensioned tie in prestressed concrete."

Leaders and followers in the development of prestressed concrete are becoming increasingly numerous. Developments in the United States during the past decade have probably been more rapid than anywhere else in the world, even though the material was relatively unused here prior to 1949. The present remarks are not intended as a complete history . . . rather they highlight some of the advances with which the writer is most familiar.

Fritz von Emperger, 1862-1942, native of Bohemia, is credited with invention of partial prestressing. He served as professor at the Technical University of Vienna, founded *Beton und Eisen* (the predecessor of *Beton- und Stahlbetonbau*) more than 50 years ago, was editor of a 20-volume encyclopedia of reinforced concrete. Dr. Emperger was elected to honorary membership in the Institute in 1932

LOS ANGELES IN '59

ACI's Technical Activities Committee is already looking forward to the 1959 convention in Los Angeles; program plans are taking shape.

Those who intend to offer papers for presentation at the 55th annual meeting should write to Institute headquarters before July 1, furnishing a synopsis which should make clear the scope of the proposed paper and indicate features that the author thinks will justify its inclusion in the program. Contributors should have preliminary drafts of manuscripts in the hands of the Technical Activities Committee for appraisal and acceptance by Sept. 15, 1958, and final manuscripts of accepted papers will be due Jan. 1, 1959.

From the replies received and suggestions from other sources, TAC will select papers to make up each convention session.

Write Institute headquarters NOW about the papers YOU want to see on the 1959 program.

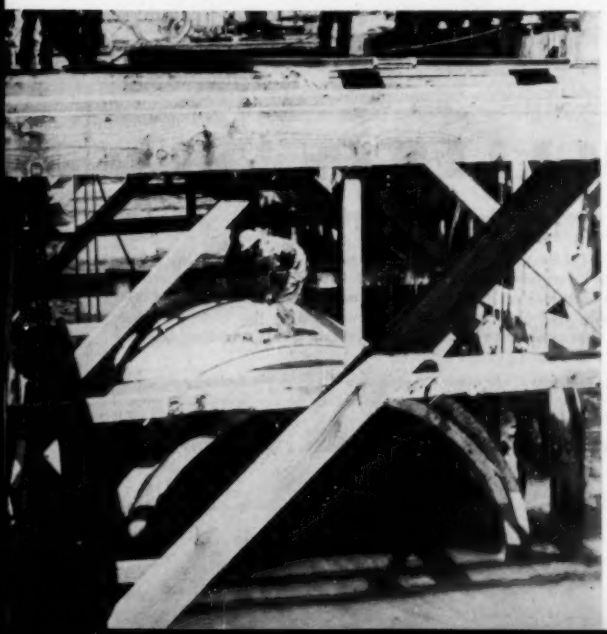
Expanded coated shale aggregate used in lightweight concrete for

Pacific Pipelines

TWO PIPELINES of reinforced concrete reach over 1800 ft into the Pacific Ocean to circulate a maximum of 352,000 gal. per min of sea water completely through the condensation system of Southern California Edison Co.'s new steam generating plant at Huntington Beach, Calif. The concrete pipelines are made of 16 ft long sections, each having a 14-ft inside diameter. There are 230 of these pipe sections used on the job, and each section weighs 50 tons. Ordinarily, sections of this size would weigh approximately 75 tons, but by using lightweight aggregate concrete, the weight was reduced a third, and approximately one-third of the cost of laying the pipe was cut. Had standard concrete been used instead of lightweight concrete, it would have meant building and using costly special equipment to lay the pipe. Savings resulting from the large cost difference between timber and steel pilings for the laying cranes was said to more than justify the additional cost of the lightweight concrete.

Lightweight mix developed

Experience gained during the famous concrete shipbuilding programs of World Wars I and II prompted the suggestion to use lightweight concrete aggregate. It had been learned that such concrete would withstand the action of sea water and possess the required permeability characteristics. After casting a number of smaller diameter lightweight sections, the pipe manufacturer developed a suitable lightweight mix and manufacturing procedure that would produce pipe meeting



Pipe being lowered into the ocean with a special hoist. Once in the water, effective weight of the giant pipe was reduced to less than 27 tons

D-load and static head tests. The concfete mix also met the other design criteria for the subaqueous pipe. These were: (1) an ultimate 28-day compressive strength of 4000 psi; (2) a density of the cured concrete not to exceed 100 lb per cu ft; (3) a drying (curing) shrinkage not to exceed 0.05 percent.

Pipe design and structural strength

The pipe diameter occasioned many related production problems. New pipe designs had to be worked out; the pipe had to be cast with a special reinforcing steel cage, and special bracket fittings for the laying cranes had to be made.

Specifically, the pipe is noncylindrical with bell-and-spigot type joints. A rubber gasket was used for closure. Two concentric steel reinforcement cages are cast into each pipe section. The structural design of the pipe provides that stress in the reinforcing steel will not exceed 12,500 psi when the pipe is subjected to its maximum internal design pressure, nor will it exceed 18,000 psi when subjected to maximum possible stresses of the combined internal pressure and external loadings.

Transportation problems

To transport the pipe from the manufacturing plant in South Gate to Huntington Beach 32 miles away, it was necessary to design a trailer which would meet California highway specifications. A rig specially built by Utility Trailer Corp., had two trailer extensions which spread the weight of the pipe section along 99 ft of road, cushioned on 42 tires, to prevent damage to the highway.

The first job in laying the pipeline was building a trestle out over the water for laying cranes big enough to handle the 14-ft diameter pipe. Then, excavation problems proved difficult. Ocean surges complicated laying the pipe beneath the ocean floor. After the pipe was lowered into the water and secured in position, sand was backfilled around it to bury the installation 5 to 8 ft below the level of the ocean floor. Once the pipe was submerged in the water, the buoyancy of the lightweight aggregate was manifest . . . the pipe's handling weight dropped from 50 tons to less than 27 tons, permitting easier positioning and handling. (Pipe composed of standard aggregate would have weighed 53 tons when submerged, almost twice the submerged weight of the pipe used.)

Lightweight concrete used in these 14 ft diameter pipes made possible a considerable saving in the cost of installation

Design and construction

The steam generating plant was designed by Bechtel Corp., in collaboration with the engineering department of Southern California Edison Co. The pipe sections were developed and manufactured by American Pipe and Construction Co., South Gate, Calif., using as lightweight aggregate a coated expanded shale manufactured by Rocklite Products, Ventura, Calif. Macco Corp., Paramount, Calif., was awarded the heavy construction contract for laying the pipe.

ACI technical committee chairman

Three new committee chairmen were appointed for 3-year terms, effective Feb. 26, 1958, with the former committee chairmen remaining as members of the respective committees.

Committee 209, Volume Changes and Plastic Flow in Concrete

William R. Lorman
U. S. Naval Civil Engineering Research and Evaluation Laboratory
Port Hueneme, Calif.

Committee 216, Fireproofing or Fire Protection of Structures

C. C. Carlson
Portland Cement Association
Chicago, Ill.

Committee 312, Plain and Reinforced Concrete Arches

Boyd G. Anderson
Ammann and Whitney
New York, N. Y.

Chairmen of the following committees, which have standards or reports pending, were reappointed for 1 year: Committee 207, Properties of Mass Concrete, headed by I. L. Tyler, Portland Cement Association, Chicago; Committee 208, Bond Stress, with C. P. Siess, University of Illinois, Urbana, as chairman; Committee 317, Reinforced Concrete Design Handbook, and Committee 323, Prestressed Reinforced Concrete, both headed by Thor Germundsson, Portland Cement Association, Chicago; and Committee 324, Precast Reinforced Concrete, Thin Sections, with chairman Arsham Amirikian, Bureau of Yards and Docks, Department of the Navy, Washington, D. C.

ACI TECHNICAL COMMITTEES will be featured in the June News Letter with an up-to-date list of all committees and members. This will supersede the committee information in the 1957 ACI DIRECTORY.

E. A. Finney, Michigan State Highway Department, East Lansing, continues as chairman of Committee 325, Structural Design of Concrete Pavements for Highways and Airports. Committee 617, Specifications for Concrete Pavements and Bases, retains chairman H. F. Clemmer, Government of the District of Columbia, Washington, D. C., and Harry Ellsberg, Giffels and Rossetti, Detroit, continues as head of Committee 622, Formwork for Concrete. L. M. Legatski, University of Michigan, Ann Arbor, will serve another year as head of Committee 711, Precast Floor Systems for Houses.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 115, Research

J. C. McCoe
Pittsburgh Coke and Chemical Co.
Pittsburgh, Pa.

Committee 212, Admixtures

M. Jack Snyder
Battelle Memorial Institute
Columbus, Ohio

R. C. Valore, Jr.
Texas Industries, Inc.
Dallas, Tex.

L. P. Witte
U. S. Bureau of Reclamation
Denver, Colo.

Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete

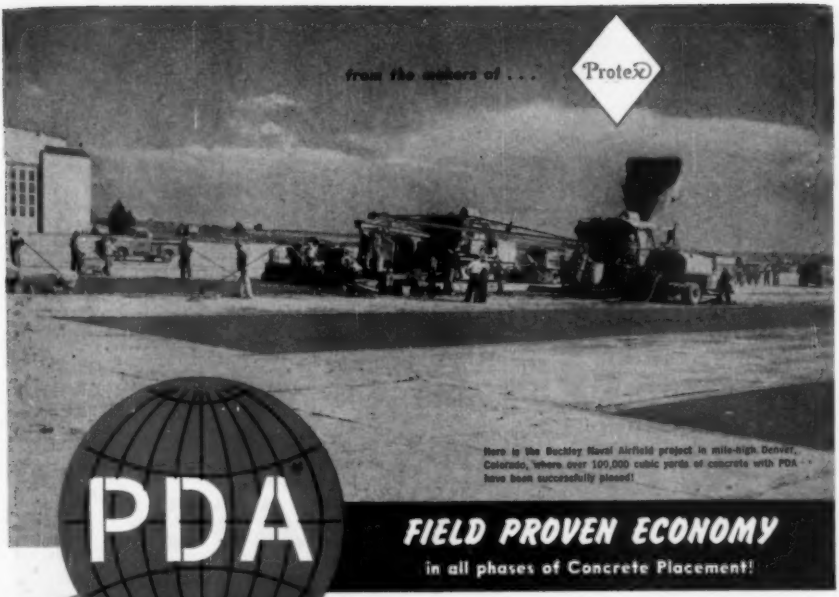
Lucas E. Pfeiffenberger
Expanded Clay and Shale Association
Alpena, Mich.

Committee 314, Rigid Frames for Buildings and Bridges

Frank Baron
University of California
Berkeley, Calif.

Richard C. Elstner
Portland Cement Association
Chicago, Ill.

Continued on p. 15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Continued from p. 13

Committee 315, Detailing Reinforced Concrete Structures

E. G. Odley
Bureau of Yards and Docks, Department of the Navy
Washington, D. C.

Committee 318, Standard Building Code

Charles S. Whitney
Ammann and Whitney
New York, N. Y., and Milwaukee, Wis.

Committee 324, Precast Reinforced Concrete, Thin Sections

Ernst Gruenwald
Lone Star Cement Corp.
New York, N. Y.

Committee 331, Structures of Concrete Masonry Units

R. O. Hedstrom
Portland Cement Association
Chicago, Ill.

Committee 332, Recommended Practice for Residential Concrete Work

Arthur Moy
City of Detroit, Department of Buildings and Safety Engineering
Detroit, Mich.

Committee 401, Specifications for Structural Concrete

John J. Manning
Concrete Industry Board
New York, N. Y.

Peter G. Keller
Specification Writer
Ottawa, Ont.

Committee 614, Recommended Practice for Measuring, Mixing, and Placing Concrete

M. R. Smith
Office of Chief of Engineers, Department of the Army
Washington, D. C.

Committee 623, Specifications and Practice for Foamed Concretes

Frank Erskine
Expanded Shale, Clay and Slate Institute
Washington, D. C.

Paul O. Freeman
Dominion Tar & Chemical Co.
Montreal, Que.

Ernst Gruenwald
Lone Star Cement Corp.
New York, N. Y.

W. C. Hansen
Universal Atlas Cement Co.
Gary, Ind.

Eugene R. Jolly
Cellular Products, Inc.
Los Angeles, Calif.

George Kalousek
Owen-Illinois Glass Co.
Toledo, Ohio

L. M. Legatski
University of Michigan
Ann Arbor, Mich.

Albert Litvin
Armour Research Foundation
Chicago, Ill.

Michael Smolin
Mearl Manufacturing Corp.
Roselle Park, N. J.

Committee 716, High-Pressure Steam Curing

D. W. Lewis
National Slag Association
Washington, D. C.

Committee 805, Application of Mortar by Pneumatic Pressure

Elmo C. Higginson
U. S. Bureau of Reclamation
Denver, Colo.

Raymond J. Schutz
Sika Chemical Corp.
Passaic, N. J.

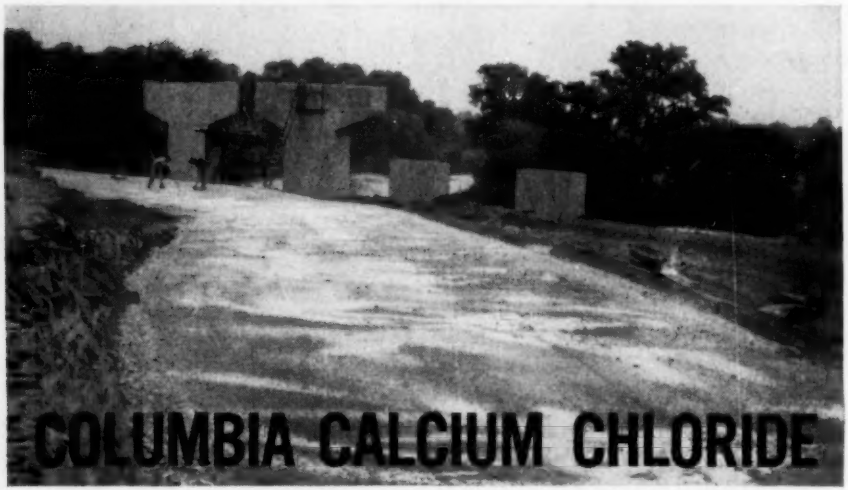
John Simmons
Pressure Concrete Co.
Florence, Ala.

R. J. Sweitzer
Lock Joint Pipe Co.
East Orange, N. J.

Max True
True Gun-All Equipment Corp.
Tulsa, Okla.

Glen Vinson
Vinson Construction Co.
Phoenix, Ariz.

Stanley G. Zynda
Gunite Contractors Association
Los Angeles, Calif.



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Positions and Projects

Bloodgood and Nielsen named by USBR

Grant Bloodgood has been named assistant commissioner and chief engineer of the U. S. Bureau of Reclamation at the Denver engineering center, and E. G. Nielsen has been appointed associate chief engineer. These appointments were effective upon the retirement of L. N. McClellan as assistant commissioner and chief engineer on Feb. 28, when he completed 46 years of service with the Bureau of Reclamation.

Both Mr. Bloodgood and Mr. Nielsen are long-time USBR career employees, Mr. Bloodgood since 1920 and Mr. Nielsen since 1934.

Mr. Bloodgood, an ACI member since 1948, received his engineering degree from the University of Nebraska in 1920. He worked for the bureau from 1920 to 1925 and then engaged in private engineering work for 4 years, returning to the bureau in 1929. His services include key assignments on the construction of the Hoover Dam, All-American Canal, and Shasta Dam. He has been associate chief engineer since January, 1954.

Mr. Bloodgood co-authored a paper with Lewis H. Tuthill entitled "Responsibilities of An Inspector" which was published in the March 1957 JOURNAL. He recently was accorded the Distinguished Service Award, the highest honor the Department of the Interior can bestow.

Mr. Nielsen obtained his BS degree in engineering from the University of Iowa in 1926. After working with private utilities and the Public Service Commission of Missouri, he joined the Bureau of Reclamation as assistant engineer in Denver in 1934. He had been assistant commissioner in Washington, D. C., since 1955.

Upson receives Cornell award

Maxwell M. Upson, chairman of the board of Raymond Concrete Pile Co., New York, recently received the first annual alumni achievement award of the Cornell Alumni Association of New York City. Presentation of the award, an illuminated scroll, was made by Deane W. Malott, president of Cornell University. Distinguished as a foundation and harbor engineer, Mr. Upson recently completed a half century of continuous service with the Raymond firm.

Mr. Upson has been an active ACI member since 1912. He was president of the Institute in 1926, has served on ACI's Board of Direction and technical committees, and has contributed several technical papers for JOURNAL publication.

ACI participates in concrete conferences on several campuses

University of North Dakota

The College of Engineering, University of North Dakota, sponsored its fifth annual concrete conference on March 12-13 on the campus at Grand Forks in cooperation with the American Concrete Institute, Portland Cement Association, Associated General Contractors of North Dakota, North Dakota Association of Architects, and the North Dakota Society of Professional Engineers. The conference was attended by over 200 persons in the construction industry interested in the production of quality concrete.

Several ACI members were among the speakers who presented talks on practical procedures, techniques, and problems of producing better concrete: Clayton L. Davis, director of technical service division, Universal Atlas Cement Co., New York, spoke on factors affecting strength results of concrete test specimens; John H. Banker, regional construction consultant, PCA, Chicago, discussed residential and industrial floor construction; Willis A. Jacus, consulting engineer, Minneapolis, spoke on "New Developments in Concrete Design and Construction—Shell and Folded Plate Roofs"; Paul F. Rice, ACI technical director, spoke on admixtures for concrete.

Montana State College

Approximately 100 individuals attended the quality concrete conference conducted Mar. 24-45, 1958, by the departments of architecture and civil engineering of Montana

State College, Bozeman, in cooperation with the American Concrete Institute, Montana Chapter American Institute of Architects, Montana Section American Society of Civil Engineers, and Associated General Contractors of America (Montana chapter). Other cooperating sponsors were National Ready Mixed Concrete Association, Montana Ready Mixed Concrete Association, and Portland Cement Association.

Among the topics discussed were fundamental properties of portland cement and concrete, concrete structures on the interstate highway program, concrete testing and construction practices, advantages of air entrainment, and the construction outlook for 1958.

University of Missouri

A structural concrete conference was conducted Apr. 11-12, 1958, at the University of Missouri, Columbia. The conference, sponsored by the university's college of engineering in cooperation with ACI and PCA, featured structural design problems in prestressed concrete, ultimate strength design, and thin shells and folded plates. Attendance totaled some 150 persons.

20% Discount

Bound Volume 55

of ACI Proceedings will cost only $6 if paid for before June 15. Regular member price is $7.50. Advance order price to nonmember subscribers is $7.20; to nonmembers who are not subscribers, $16.80; regular prices are $9 and $21, to the latter two, respectively.

ACI Publications Department

A number of ACI members were speakers on the program including: A. A. Brielmaier, Washington University, St. Louis, who presented the opening technical talk of the conference on basic principles of prestressed concrete; E. J. Critzas, Missouri Prescrete, Inc., Overland, spoke on production of prestressed concrete followed by Adrian Pauw and John E. Breen, both of the University of Missouri, describing a testing program for a prestressed concrete bridge.

The afternoon session on April 11 was presided over by Sabri Sami, University of Missouri. At this assembly Leo Corning, Portland Cement Association, Chicago, spoke on "Ultimate Strength Design of Reinforced Concrete—Why and How." Paul Rice, American Concrete Institute, based his talk on the ultimate strength design and the new ACI Building Code.

Ivan Viest, American Association of State Highway Officials Road Test, Ottawa, Ill., spoke on shear strength of reinforced concrete without web reinforcement, and Narbey Khachaturian, University of Illinois, Urbana, presented "The Relationship Between Design Criteria at Working Loads and at Ultimate."

Milo Ketchum, Ketchum and Konkel, Denver, opened the April 12 session with a talk on folded plate construction, followed by Dutton Biggs, Portland Cement Association, Kansas City, who discussed progress in thin shell construction.

Hennegar elected Maguire vice-president

G. H. Hennegar was recently elected vice-president of Walter Maguire Co., Inc., New York. He joined the firm in 1950 and was formerly assistant to the president in charge of sales and distribution. Mr. Hennegar is a graduate of the University of Illinois and attended the Harvard Graduate School.

Lipkind named chief chemist

Henry Lipkind has been appointed chief chemist at the Belleville, N. J., plant of L. Sonneborn Sons, Inc. Mr. Lipkind joined Sonneborn in 1946.

A graduate of New York University, he pursued graduate studies at the University of Minnesota, Iowa State College, and Polytechnic Institute of Brooklyn. During extensive travels throughout the United States and abroad he has lectured on materials for construction and maintenance, and holds patents in the fields of silicones and cements. Mr. Lipkind has been an ACI member since 1952.

Proposed world construction program

An exploratory conference held at Northwestern University, Evanston, Ill., has confirmed plans for a World Construction Program. As outlined by the temporary chairman of the conference, Howard T. Fisher, Chicago architect and visiting professor at Northwestern, the basic objective of the program is to advance technical progress and sound public relations in the construction industry.

Some 300 engineers, architects, educators, and constructors attended the planning meeting in December. As the result of preliminary discussion, the group envisioned a program extending over a 5-year period with a World Construction Congress during 1961 which would review and appraise the activities carried out.

Sponsors of the meeting were the American Institute of Architects, the American Society of Civil Engineers, and Northwestern University Technological Institute.

Sika organizes Canadian affiliate

Sika Chemical of Canada, Ltd., a wholly owned affiliate of Sika Chemical Corp., Passaic, N. J., has been formed. The corporation's officers are: F. A. Schenker, president; Emil Schmid, vice-president; A. G. Clauson, treasurer; and Stanley J. Moore, manager. The new company's main officers are in Toronto.

Milsom elected Exchange president

Elected as 1958 president of The Builders' Exchange at its 60th annual meeting held in Montreal, Gerald Milsom has had many years of experience as an officer and member of the exchange.

Mr. Milsom has been in the construction industry since 1927. In 1946 he organized Metallicrete Floor Co., Ltd., a firm of concrete floor specialists, of which he is president. Active in a number of civic and professional organizations, Mr. Milsom currently is chairman of ACI Committee 402, Concrete Floor Finishes.

Wayne Woolley

Wayne R. Woolley, civil engineer for Truscon Steel Division of Republic Steel Corp., died March 11 in Youngstown, Ohio.

Mr. Woolley, a graduate of the University of Illinois, was a member of the Highway Research Board and the American Society for Metals. He became an Institute member in 1946.

Joseph Sterling Kinney

Joseph Sterling Kinney, head of the structural engineering division at Rensselaer Polytechnic Institute, Troy, N. Y., died January 29. He had devoted most of his career to service at Rensselaer.

Dr. Kinney received his undergraduate and graduate degrees at RPI, but did some graduate work in structures at the University of Michigan. He was a design engineer with American Bridge Co. for several years and did some consulting work, but his primary interest was teaching. Dr. Kinney joined ACI in 1947.

Walter H. Weiskopf

Walter H. Weiskopf, prominent consulting structural engineer, died January 26 in New York. A partner in Weiskopf and Pickworth, New York, he participated in the design of numerous well known buildings.

Mr. Weiskopf worked 9 years in the design departments of U. S. Steel and Carnegie Steel companies before joining Weiskopf and Pickworth. Recent structures for which he served as consultant include the 52-story Union Carbide building and the 60-story Chase Manhattan Bank building now under construction in New York; Lever House; the new United States Senate office building; Manufacturers Trust Co. bank building in New York; and the Ford Motor Co. staff office building in Dearborn, Mich.

During World War II, Mr. Weiskopf served as head engineer of the New York District, Corps of Engineers. He was a 1920 civil engineering graduate of Rensselaer Polytechnic Institute, and had been an Institute member since 1946.

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Travel grants for scientists

The National Science Foundation is authorized to provide a limited number of travel grants for the purpose of assisting qualified scientists who wish to attend certain conferences, congresses, and other meetings of scientists held abroad which are concerned with research in the sciences. Decisions will be made by the foundation concerning specific meetings for which travel support can be extended.

Physical scientists, mathematicians, and engineers who wish support to attend international meetings concerned with subjects in the *physical sciences* should submit application forms for financial assistance prior to June 30 for travel between October 1 and March 31, and prior to December 31 for travel between April 1 and September 30. Requests for application forms should be addressed to Assistant Director for Mathematical, Physical, and Engineering Sciences, National Science Foundation, Washington 25, D. C.

Hercules, Peerless and Riverside merge

The American Cement Corp. has been organized as the result of the merger of Hercules Cement Corp., Philadelphia, Peerless Cement Corp., Detroit, and Riverside Cement Co., Los Angeles. Garner A. Beckett was named chairman of the board of directors, and D. S. MacBride becomes president; main offices of the company are in Philadelphia.

In keeping with basic policy of American Cement Corp., the three divisions will operate as substantially independent units, continuing much the same pattern they maintained as separate corporations.

Gutt named vice-president

Tadius J. Gutt has been named vice-president and general manager of George Rackel & Sons Co., Cleveland, structural concrete manufacturer. Mr. Gutt, formerly general manager of the Texas Stressed Concrete Corp., Austin, joined the Rackle firm as assistant to the president in 1956. A graduate of the University of Rochester, he became a member of ACI in 1949.

Straub awarded ASCE research prize

Dr. Lorenz G. Straub, director of the St. Anthony Falls Hydraulic Laboratory and head of the department of civil engineering, University of Minnesota, was awarded the American Society of Civil Engineers 1957 Research Prize.

An ACI member since 1926, Dr. Straub, well-known engineering consultant in hydraulics, has taken an active part in the work of many leading professional and scientific societies here and abroad, and has published innumerable articles, papers, and books. Among his many distinguished special honors in the last few years are the President's certificate of appreciation for wartime services, an award for ordnance development by the Department of the Navy, and the French award of "Officer d'Academie" and the "Order of Palms."

Martin named president of Louisville Cement

Louisville Cement Co., Louisville, Ky., recently named Boyce F. Martin as president and chief executive officer. He succeeds Eugene D. Hill who was elected chairman of the board.

John J. Mallon was elected to the new position of senior vice-president and Homer Baker was made a vice-president. Harold Irick succeeds Martin as treasurer. Other officers of the company are vice-presidents R. K. Hartsock and David E. Willingham and Roland Whitney, who is secretary. Robert Gibson and Frank Lovell were named sales managers in other organizational changes.

Allied Chemical assumes new name

One of America's largest diversified chemical companies, in business since 1920 as Allied Chemical & Dye Corp., effective May 1 became the Allied Chemical Corp. According to Glen B. Miller, company president, the directors believe this new name will more clearly reflect the broad nature of the company's position in the chemical industry and will not single out a particular division or field of activity. Allied's six operating divi-

sions employ about 30,000 employees who produce some 3000 products at more than 120 plants, 12 research laboratories, mines, and quarries.

Ross appointed research director

Hugh C. Ross has been appointed director of research of the Hydro-Electric Power Commission of Ontario, Toronto, succeeding Gordon B. Tebo. Mr. Ross has a long record of continuous service with the research division, having joined the staff in 1929 immediately after graduation from the University of Toronto.

A member of ACI since 1931, Mr. Ross has been active in Institute affairs for many years. In 1954 he served on the Board of Direction and presently is a member of ACI Committee 115, Research, and Committee 613, Recommended Practice for Proportioning Concrete Mixes. He also takes an active part in ASTM technical committee work and serves on various committees of the Canadian Standards Association, particularly those pertaining to concrete and other construction materials.

Building Committee Report

as of March 31, 1958

Most recent report of the ACI Building Committee shows the following contributions to the Building Fund, arranged by categories.

Cement producers	$ 76,802.00
Reinforcing steel industry	40,000.00
Engineers and architects in private practice	34,206.22
Contractors	19,570.00
Ready-mixed concrete and aggregate industry	12,548.87
Membership at large	9,199.33
Admixtures manufacturers	7,526.83
Concrete machinery and specialty products	3,585.00
Concrete products manufacturers	2,595.00
Special contributions	1,766.43
Eastern Canada	1,496.90
	$209,296.58
Pledges in hand	2,760.00
GRAND TOTAL	$212,056.58

Parme made bureau manager

The appointment of Alfred L. Parme as manager and Walter E. Kunze, Jr., as assistant manager of the structural and railways bureau of the Portland Cement Association, Chicago, was recently announced. Mr. Parme succeeds Thor Germundsson who will continue with PCA in the capacity of a staff advisor.

Author of several JOURNAL papers, Mr. Parme has contributed to the development of valuable data in the design of storage tanks, rigid frame bridges, arch dams, shells, and on the ultimate strength theory of design. Active in ACI technical committee work he is chairman of ACI-ASCE Committee 328, Limit Design, and a member of ACI-ASCE Committee 326, Shear and Diagonal Tension; he is also a member of ACI Committee 314, Rigid Frames for Buildings and Bridges, and ACI Committee 334, Concrete Shell Structures. Mr. Parme graduated from Cornell University in 1935. He was engineer for the Phoenix Engineering Division of Ebasco Services, New York, and for the Corps of Engineers in Binghamton, N. Y., prior to joining PCA in 1940.

Mr. Kunze joined the association as a structural engineer in the structural and railways bureau in 1952. He is a graduate of The Citadel, Charleston, S. C., with a degree in civil engineering, and of Massachusetts Institute of Technology where he was awarded his MS degree in structural engineering in 1950. Before joining PCA he served with the consulting engineering firms of Metcalf & Eddy in Boston, and Toltz, King & Day in St. Paul.

Carpenter elected Chain Belt president

O. W. Carpenter has been elected president of Chain Belt Co., Milwaukee, succeeding L. B. McKnight who had been president of the company since 1953. Mr. Carpenter came to Chain Belt in 1943 and has served in various executive capacities with the company since then.

Mr. McKnight will continue as a director of the firm and will also serve as chairman of the executive committee of the board of directors and as a consultant to the management.

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   - A. Minimize shrinkage in concrete before and after hardening, due to lower water content and more complete hydration of cement.
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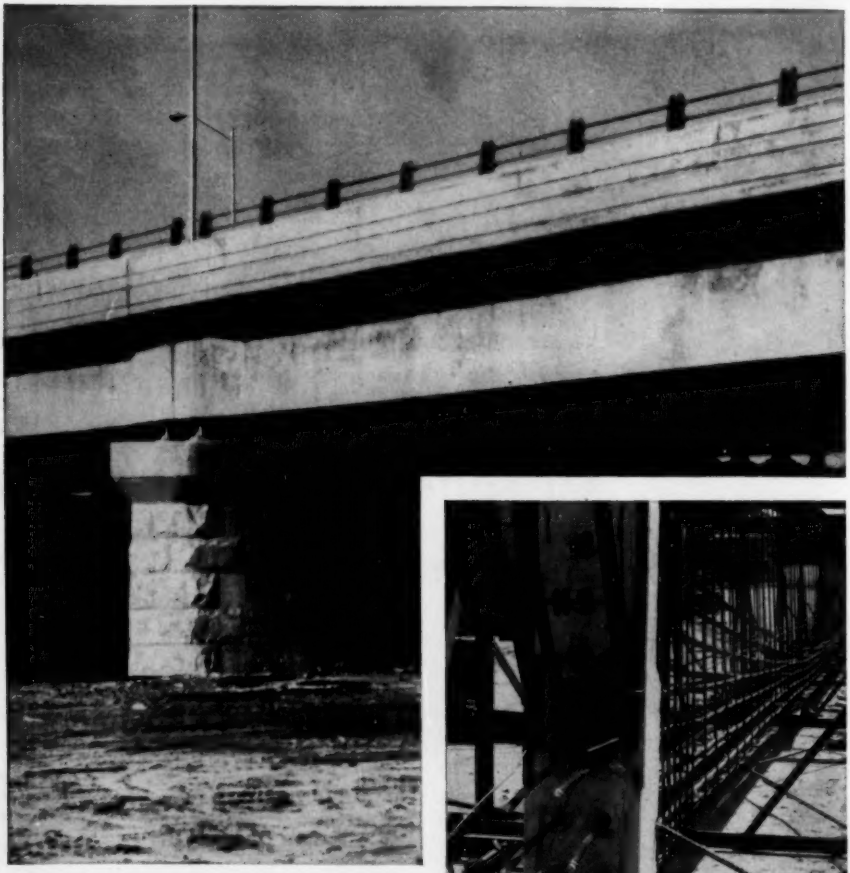
On the Wolcott Avenue Bridge



Fabrication of prestressed concrete for the Wolcott Avenue Bridge was made much simpler through the use of *new* USS American Super-Tens Stress-Relieved Wire. Super-Tens is unusually *straight* prestressing wire. It stays straight . . . lies flat. This development has solved a major problem facing the manufacturer of prestressed concrete.

This is a partially completed reinforcing assembly for girders in the Wolcott Avenue Bridge. Super-Tens Wires have been inserted in six of the twelve flexible metal tubes in this particular assembly.

These are the people who built this bridge:

Owner:	The Greater Hartford Bridge Authority
General Contractor:	Merritt-Chapman & Scott Corp., New York
Designing Engineer:	Thomas Worcester, Inc., Boston
Supervising Engineer:	Deleuw, Cather & Brill, New York
Consulting Engineer for Contractor:	The Preload Co., Inc., New York

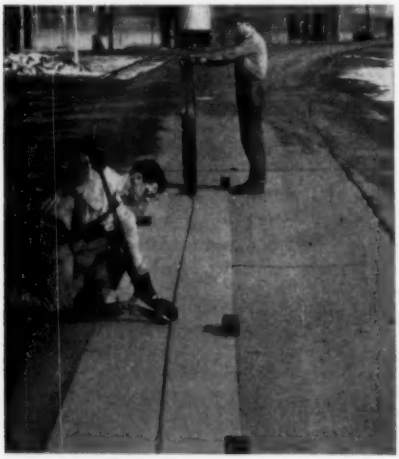
extraordinary straightness of new

USS American Stress-Relieved Wire

solves on-site prestressing problem

120-Foot Beams Prestressed Without Difficulty

The Wolcott Avenue Bridge, near Hartford, Connecticut, is a new cast-in-place, slab and girder bridge construction. It is built of concrete girders post-tensioned on the job with an amazing new product, USS American Super-Tens Stress-Relieved Wire.



This photo emphasizes the superior straightness of USS American Super-Tens Stress-Relieved Wire as it is uncoiled. It stays straight and lies flat, with no tendency to curl, which greatly simplifies handling.

This wire has straightness and handling ease never before realized in the field of prestressed concrete. New Super-Tens Stress-Relieved Wire developed by American Steel & Wire eliminates the tendency of wire to return to its original shape. This tendency has been a major problem in making up parallel-wire, post-tensioning assemblies. By using new, *straight* Super-Tens Stress-Relieved Wire, however, the builders of the Wolcott Avenue Bridge not only saved time on the prestressing operation, but did the job more efficiently.

Spans are 120 feet long; girder webs are seven feet deep and twelve inches wide. The Freyssinet post-tensioning system was used. Each cable contains twelve 0.276-inch-diameter USS American Super-Tens Stress-Relieved Wires.

Perfecting a straighter prestressing wire is the latest milestone for American Steel & Wire engineers, leaders in the application and development of steel wire and strand for prestressed concrete. If you are interested in prestressed concrete construction, get in touch with us today through our nearest Sales Office. Or write to American Steel & Wire, Rockefeller Building, Cleveland 13, Ohio.

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Annual award established for paper on foundations

An annual award in honor of Alfred A. Raymond, inventor of the cast-in-place concrete pile, has been established by Raymond Concrete Pile Co., New York. The award, carrying a prize of $1000 will be presented yearly to the author of the best original paper on any aspect of design and construction of foundations for structures.

The purpose of the award is to encourage original research in the field of foundation engineering. Manuscripts, acceptable from practicing and professional engineers, engineering faculty members, and graduate students, must be in English, limited to 2500 words, and in the hands of the judges by Sept. 1, 1958 for the first year's competition. The successful paper will be selected by a panel of three judges: Ralph B. Peck, professor of foundation engineering, University of Illinois, Urbana; R. E. Fadum, head, department of civil engineering, North Carolina State College of Agriculture and Engineering, Raleigh; and E. A. Dockstader, consulting engineer, Boston.

Further information and complete instructions may be secured from Alfred A. Raymond Award, Room 1214, 140 Cedar St., New York 6, N. Y.

BRI holds seventh annual meeting

The Building Research Institute conducted its seventh annual meeting in Washington, D. C., April 21-23, 1958. BRI is a unit of the division of engineering and industrial research of the National Academy of Sciences, National Research Council. It is a private, nonprofit organization of scientists dedicated to the furtherance of science and its use for the general welfare.

Topics discussed at the conference included world-wide trends in building research, fire protection of buildings, new materials information, cost-saving developments, and future building programs of the federal government.

Two ACI members appeared as speakers at the meeting: Charles H. Topping, senior architectural and civil consultant, E. I. du Pont de Nemours & Co., Inc., Wilmington, Del., BRI president, who gave the BRI annual activities report; and Douglas E. Parsons, chief, building technology division, National Bureau of Standards, who served as chairman of the session devoted to world-wide trends in building research.

Gibbons joins Alpha sales force

Joseph P. Gibbons has joined Alpha Portland Cement Co., Easton, Pa., as a sales representative in northeast Pennsylvania. For the past 13 years Mr. Gibbons had been general manager of the Scranton Builders Supply Co.

Murlin opens engineering office

John A. Murlin recently organized Murlin Engineering, structural design consultants, with offices in Dallas, Tex. E. D. Mayes will be an associate of the firm. Both Mr. Murlin and Mr. Mayes are ACI members.

Mr. Murlin is a graduate of Massachusetts Institute of Technology, with experience in aircraft structures, hydraulic machinery, production control, acoustics, and heat transfer. At the present he is a member of ACI Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete.

Mr. Mayes, structural engineer and architect, has had extensive experience in structures work in the United States and in Spain.

Errata

The following correction should be made in "Some Physical Properties of Concrete at High Temperatures," by Robert Philleo, which appeared in the April 1958 JOURNAL.

p. 862—In Table 1, second line of data from the bottom, fourth column should read 4.2×10^{-6}. Fourth and sixth columns of the last line of data should read 5.4×10^{-6} and 4.9×10^{-6}, respectively.

* * *

The following correction should be made in "Specific Surface of Aggregates Related to Compressive and Flexural Strength of Concrete," by B. G. Singh, which appeared in the April 1958 JOURNAL.

p. 903—Fig. 6, caption should read "ϕ_c^1 plotted against strength at 28 days . . . "

p. 903—Fig. 7, caption should read "ϕ_c^1 plotted against strength at 7 days . . . "

p. 907—Line 9 should read "7-day test results than in Fig. 6 giving the 28-day results. Further, if this "

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ASTM conducts committee week, mortar symposium

The American Society for Testing Materials held its annual committee week in St. Louis, Feb. 10-14, 1958. Thirty-five of ASTM's technical committees held some 280 committee and subcommittee meetings during the week, with 1135 technical men registered to discuss and correlate the research upon which ASTM specifications and methods of test are based. The committees put into final shape new and revised tentatives and standards which will be recommended to the society for adoption at its annual meeting in Boston in June.

Symposium on mortars

Workability, and the proper selection of materials were stressed as the two most important factors of good masonry mortar at a symposium sponsored by ASTM Committee C-12 on Mortars for Unit Masonry. The essential properties of a good mortar were outlined by John W. McBurney, materials consultant, Silver Spring, Md., formerly with the National Bureau of Standards and a long-time member of Committee C-12. Mr. McBurney referred to a presentation by J. C. Pearson in 1939 which listed in the order of their importance the essential properties of mortar. Workability was first and Mr. McBurney believes that this is still the most important property. The second, soundness, has now been corrected to a large degree by the use of pressure lime. Bond strength is still a live subject. Strength, both tensile and compressive, which ranked ninth in 1939, remains one of the lesser properties since load bearing masonry walls are not being used as much as formerly.

A method of determining bond strength was described by Cyrus C. Fishburn, research associate, National Bureau of Standards, Washington, D. C., and the elasticity of mortar and its implications were discussed by Walter C. Voss, consultant on architectural construction and materials and formerly head of the building engineering and construction department of Massachusetts Institute of Technology. Professor Voss stressed the need for a low modulus mortar to decrease rigidity and the need for frequent relief joints where high strength mortar is required. He admonished the engineer to cease idolizing strength and pay more attention to the selection of the components, particularly in reference to climatic exposures.

A progress report on research on the durability of mortar being conducted at Lafayette College was presented by W. L. Zematis, instructor of testing materials. This research program is being sponsored by Alpha Portland Cement Co., Easton, Pa. Data were presented showing the effect of entrained air on the freezing and thawing test and it was indicated that lime is the most important constituent in mortar in respect to durability.

Two short papers were presented on the effect of sand grading upon mortar properties. C. U. Pierson, Jr., technical director, Southern Cement Co., Birmingham, Ala., reported on bond test studies in his laboratory. The need for improvement in sand gradings was expressed as comparisons of 15 different sources of sand were given. The tensile bond test was proposed as a criterion in establishing

The ACI JOURNAL is still a good buy!

Rates increased Mar. 1, 1958, to $18 per year for nonmember subscribers, both foreign and domestic. ACI members continue to receive the JOURNAL as one of the considerations of dues payment.

proper sand gradings. Walter Washington, Sumner Sollitt Co., Chicago, discussed various gradings of a sand in mortar mixes, presenting data on the behavior of sand with gradings not meeting ASTM specifications. The sand cone method (ASTM C 128) was found not entirely reliable in establishing the saturated surface-dry condition of sand. Mr. Washington stated that sand with unusual gradings, normally considered poor, might be satisfactory if the grading is such that voids are low and the fineness modulus is in a suitable range.

The results of tests of bond between masonry grout and reinforcing steel were reviewed by William Lerch, Portland Cement Association, Chicago. He reported that mortars made with different masonry cements prepared in accordance with two ASTM specifications for mortar (C 161 and C 270) developed bond strengths which appeared adequate for reinforced brick masonry construction. Harry C. Plummer, Structural Clay Products Institute, Washington, D. C., presided as symposium chairman.

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ACI *Proceedings* V. 37

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ACI Publications Department

Committee C-12 action

Publication of a second and possibly final report on efflorescence was approved at the meeting of Committee C-12 during the ASTM committee week. This report contains extensive data gathered by the subcommittee on research, and will be published in an early issue of the *ASTM Bulletin*. The subject of false set is also being considered by the subcommittee with a cooperative test series planned jointly with Committee C-1 on Cement.

Committee C-12 officers are: chairman, R. E. Copeland, National Concrete Masonry Association, Chicago; first vice-chairman, Harry C. Plummer, Structural Clay Products Institute, Washington, D. C.; second vice-chairman, P. M. Woodworth, The Waylite Co., Chicago; and secretary, C. U. Pierson, Jr., Southern Cement Co., Birmingham, Ala.

Committee A-1 on steel

Standards for steel for concrete reinforcement continue to receive much attention from ASTM Committee A-1. In 1957 two new specifications, one for sizes 14S and 18S large deformed billet steel bars (A 408) and another for uncoated 7-wire strand for prestressed concrete (A 416) were published. A new specification for uncoated wire for prestressed concrete will be submitted to ASTM for adoption at the annual meeting in June, 1958. Work also is being started on a new specification for high-strength billet steel reinforcement with a yield strength of 75,000 psi.

Revisions will also be proposed for adoption for basic billet steel, rail steel, and large size billet steel specifications to change the yield point for the hard grade from the present required minimum of 50,000 psi to 60,000 psi. "Building Code Requirements for Reinforced Concrete (ACI 318-56)" adopted by the American Concrete Institute already recognizes these changes.

World engineering assembly in Israel

The World Assembly of Engineers and Architects, Friends of Israel, will be held from May 29 to June 9, 1958, at Tel Aviv, Haifa, and Jerusalem, in conjunction with the celebration of the tenth anniversary of the establishment of Israel. The meeting is being sponsored by the Association of Engineers and Architects of Israel and is endorsed by the Technion—Israel Institute of Technology, Israel's only engineering university.

Engineers and architects from all parts of the world, as well as from Israel itself, are expected to participate in sessions and seminars whose central theme is the role of technology in the development of Israel. Among the outstanding engineers who will address the group is ACI member, William Ginsberg, consulting engineer, New York, who will speak on the planning of industrial plants.

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. The grades of membership are described overleaf.

All who have an interest in concrete are eligible for membership.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI JOURNAL provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

Opportunity for service is present in technical committee activity, in contributions, or only comments, to the ACI JOURNAL, or in reviewing technical publications for material of interest to the membership.

ACI's world-wide membership is growing in extent and participation—traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

(cut here)

Board of Direction, American Concrete Institute
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Of the annual dues, $12.00 is for the JOURNAL of the American Concrete Institute (except that dues for Junior and Student Members apply in full for the JOURNAL).

The undersigned hereby applies for_____**membership**
(Individual, Corporation, Contributing, Junior, Student)

in the American Concrete Institute. Proposed by_____

For Corporation Membership, ACI representative will be_____

_____ _____
(Date of graduation if Student) (Name, if Corporation)

Signature_____

For our records, please complete both sides of application.

EXCERPTS FROM BYLAWS: ARTICLE I—MEMBERS

Section 1. This Institute shall consist of Members, Corporation Members, Contributing Members, Junior Members, Student Members, and Honorary Members interested in furthering the Institute's objects as set forth in its Charter.

Sec. 2. A Member shall be an individual.

A Corporation Member shall be a firm, corporation, society, agency of government or other organization. A Corporation Member shall name one individual as its representative who will enjoy all membership rights and privileges.

A Contributing Member shall be an individual, firm, corporation, society, agency of government, or other organization wishing to give larger support to Institute activities through the payment of larger dues. Contributing Members, other than individuals, shall name representatives as do Corporation Members.

A Junior Member shall be a person less than 28 years old.

A Student Member shall be less than 28 years old and a student in residence at a recognized technical or engineering school.

An Honorary Member shall be a person of eminence in the field of the Institute's interest, or one who has performed extraordinarily meritorious service to the Institute.

Sec. 3. All classes of Members except Honorary Members and Student Members, shall be proposed by at least one Member of the Institute and elected by a two-thirds vote of the Board of Direction. An Honorary Member shall be elected by unanimous vote of the Board of Direction. A Student Member shall be proposed by a member of his school's faculty, who need not be a Member of the Institute, and elected by a two-thirds vote of the Board of Direction.

Sec. 4. All Members shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall not vote nor hold office. The status of a Student Member shall change automatically to that of Junior Member on the first anniversary of his membership succeeding the date on which he ceases to be a student in residence. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

Sec. 5. Applications for and resignations from Membership and requests for change of representatives of Corporation or Contributing Memberships shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from Members whose dues are not more than 60 days in arrears, except by special action of the Board of Direction.

(cut here)

Name _____
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Who's Who This Month

ACI Committee 324

"Tentative Recommendations for Thin-Section Reinforced Precast Concrete Construction," appearing on p. 921 is the report of ACI Committee 324, Precast Reinforced Concrete, Thin Sections, headed by Arsham Amirikian, Bureau of Yards and Docks, Department of the Navy, Washington, D. C.

The eleven-man committee includes: Arthur R. Anderson, consulting engineer, Tacoma, Wash.; Boyd G. Anderson, Ammann and Whitney, New York; K. P. Billner, Vacuum Concrete, Inc., Philadelphia; Louis P. Corbetta, Corbetta Construction Co., Inc., Chicago; and George P. Duecy, Associated Sand and Gravel Co., Everett, Wash.

Ernst Gruenwald, Lone Star Cement Corp., New York City; Daniel P. Jenny, Portland Cement Association, Washington, D. C.; Folmer Jorgensen, civil engineer, Copenhagen; Ezra G. Odley, Bureau of Yards and Docks, Department of the Navy, Washington, D C.; C. D. Wailes, Jr., Wailes Precast Concrete Corp., Sun Valley, Calif.; and H. J. Wolbeer, N. V. Schokbeton, Zeist, Netherlands, also serve on Committee 324.

Frank W. Chappell

Frank W. Chappell, Dallas consulting engineer, describes the design of a multistory building built entirely of lightweight concrete and presenting unusual problems in the design of columns for lateral stability as well as the support of vertical loads. His paper, "High Strength Steel and Concrete Result in Minimum Column Sizes," appears on p. 929.

Mr. Chappell's experience in the design of reinforced concrete dates back to a hollow concrete dam 90 ft high which was built in the middle twenties at Cisco, Tex. Since that time he has been responsible for many concrete structures including the famous Cotton Bowl at Dallas. During World War II he was resident partner of a firm which designed a large defense project requiring more than 500,000 cu yd of concrete.

A graduate of Vanderbilt University, he taught 2 years in the school of engineering of the University of Oklahoma before coming to Texas. He is a member of ACI, ASCE, and the National Society of Professional Engineers.

Alfred L. Miller

"Warping of Reinforced Concrete Due to Shrinkage" appearing on p. 939 is based on experimental research by Alfred L. Miller, professor of mechanics and structures, department of civil engineering, University of Washington, Seattle. A graduate of that university, he was appointed to the faculty in 1923, he has served continuously in engineering mechanics, structures, and structural materials. In addition to teaching and research, Professor Miller has engaged in extensive consultation work with the engineering and architectural professions, technical associations, and public agencies in the design and analysis of structural systems, the properties of structural assemblies, specifications, and building codes.

Professor Miller has participated extensively in the development of present methods of analysis and design of continuous structures, particularly with regard to lateral and dynamic loads. His work with concrete includes pioneer studies of steam curing, the effect of high temperature on portland and high alumina cements, and stresses in continuous structures due to thermal differentials. He became a member of the Institute in 1946.

F. W. Cox and J. L. Ennenga

Results of experiments conducted by Francis W. Cox and James L. Ennenga both of the Corps of Engineers, Omaha district, appear in "Transverse Strength of Concrete Block Walls" on p. 951 of this issue.

Mr. Cox attended Iowa State College and the University of Illinois, receiving degrees in civil engineering and sanitary engineering from these institutions. Prior to World War II he was employed by the Shell Oil Co., Iowa State College, and the architectural firm of G. Meredith Musik, Denver. In 1946 he joined the Omaha District, Corps of Engineers. Since 1952 he has been head of the structural design section of the military branch. In this

LOOKING AHEAD

June 16-20, 1958—American Society for Engineering Education, Annual Meeting, University of California, Berkeley, Calif.

June 22-28, 1958—American Society for Testing Materials, 61st Annual Meeting and Exhibit, Hotel Statler, Boston, Mass.

June 23-27, 1958—American Society of Civil Engineers, Portland Convention, Multnomah Hotel, Portland, Ore.

Sept. 15, 1958—6th International Congress on Large Dams, New York, N. Y.

Sept. 21-25, 1958—Prestressed Concrete Institute, Fourth Annual Convention, Edgewater Beach Hotel, Chicago, Ill.

Oct. 27-29, 1958—American Concrete Institute, Regional Meeting, Statler Hotel, Detroit, Mich.

Nov. 18-20, 1958—American Standards Association, Ninth National Conference on Standards, Hotel Roosevelt, New York, N. Y.

capacity he is responsible for the structural integrity of the many buildings designed for army and air force bases within the 6-state area of the Omaha district. Mr. Cox became a member of ACI in 1956.

Mr. Ennenga graduated in civil engineering from South Dakota School of Mines and Technology in 1947. In 1949 he was employed by the Corps of Engineers to help design the concrete structures for Garrison Dam. Since then he has worked almost continuously with concrete structures, both in design and construction. He is now head of the structural design group, military branch, Omaha District Engineer's Office, Corps of Engineers.

Fritz Kramrisch

"Reinforcement of Press Foundations by Post-Tensioning" on p. 961 is the solution to a practical problem prepared by Fritz Kramrisch, senior structural engineer with Albert Kahn Associated Architects and Engineers, Inc., Detroit. Mr. Kramrisch graduated in 1933 in civil engineering from the Technical University in Vienna, Austria, and received a doctor's degree from the same university in 1935. From 1935 to 1940 he worked as structural designer with various firms in Austria and England.

In 1940 Fritz Kramrisch joined the Albert Kahn organization in Detroit, where he has been engaged primarily with the structural design of industrial buildings, office buildings, and hospitals. In 1953 he designed a multistory office building for the Glenn L. Martin Co., Baltimore, partly prestressed and entirely precast concrete, one of the first buildings of this type in the United States.

An ACI member since 1955, he is presently a member of ACI Committee 322, Design of Structural Plain Concrete. He is also a member of the American Society of Civil Engineers and other professional organizations.

Harold S. Davis

Originally presented at the ACI regional meeting in Seattle last November, "High-Density Concrete for Shielding Atomic Energy Plants" by Harold S. Davis, now appears in print for JOURNAL readers on p. 965. Mr. Davis is with the Hanford Atomic Products Operation, General Electric Co., Richland, Wash.

A graduate of the University of Idaho in 1940, he received his MS degree in 1947 and PhD in 1949 from Northwestern University. Graduate studies were in the fields of structures, foundations, and hydraulics.

He has taught civil engineering at the University of Idaho, Northwestern University, and the State College of Washington. He also taught in U. S. Navy training schools during Wold War II. He has had summer employment with a number of private industries including the Idaho Materials Testing Laboratory, Moscow, Idaho; Whitehouse and Price, architects and engineers, Spokane; and Boeing Airplane Co., Seattle.

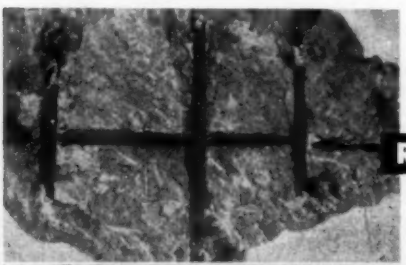
Mr. Davis joined General Electric Co. at Richland in 1951. He supervised a developmental program on high-density concrete, and the design of reactor shields and associated

structures. He has been a member of ACI since 1951, and was co-author of "Properties of High-Density Concrete Made with Iron Aggregate" which appeared in the ACI JOURNAL, March 1956.

Inge Lyse

"Tests of Full-Size Prestressed Concrete Bridge Beams" on p. 979 was submitted by Inge Lyse, professor of reinforced concrete and director of the concrete laboratory at the Norwegian Institute of Technology, Trondheim. Graduating in 1923 from the Norwegian Institute of Technology in civil engineering, he immediately left for Los Angeles in search of employment. There he joined the Southern California Edison Co. and was assigned to the construction work at Big Creek. In 1925 he became a member of the testing staff at the Stevenson Creek Experimental Dam and served under the late W. A. Slater until the completion of the report in 1927. He then joined the Portland Cement Association doing research work in the fields of concrete aggregates, permeability and durability of concrete. In 1929 he was transferred to Lehigh University as the PCA representative on the investigation of reinforced concrete columns under the direction of the ACI committee on columns (then designated Committee 105). In 1931 he was appointed research assistant professor of engineering materials at Lehigh University later being placed in charge of the Fritz Engineering Laboratory. In 1938 he left Lehigh University in order to become professor of reinforced concrete at his alma mater.

Professor Lyse has been a member of ACI since 1926. He served on the Board of Direction of the Institute in 1937-38. He is the author or co-author of more than 80 research papers, mostly in the field of concrete, many of which have appeared in the ACI JOURNAL. Last year he published with Mr. Wiig as co-author a textbook in plain and reinforced concrete (in Norwegian). During the summer of 1957 he and Franklin R. McMillan made an extensive survey of concrete dams in the United States for the Portland Cement Association under the sponsorship of ACI Committee 207, Properties of Mass Concrete.



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Professor Lyse is now president of the Norwegian Concrete Institute and of the Norwegian Academy of Technical Sciences. He is also the chairman of the Norwegian committee for concrete building code.

Valerian Leontovich

"Concept of Elastic Parameters," p. 987, has been developed by Valerian Leontovich, senior structural engineer of the International Engineering Co., Inc., San Francisco. The Russian-born author received his degree in civil engineering from the Harbin Polytechnic Institute in 1927 and subsequently in 1947, his MS degree in civil engineering from the University of California.

The first decade of his career, Mr. Leontovich spent in the Far East and Philippines, as a structural designer and a professional engineer. He came to the United States before World War II, and has worked on a number of impressive projects during the past 20 years.

In 1942, being with Ellison and King, consulting engineers of San Francisco, he was charged with responsibility of directing the design of 10,000-ton capacity reinforced concrete barges built for the U. S. Maritime Commission. As a principal designer of Pacific Islands Engineers, Inc., he was a strong advocate of rigid frame construction for military installations in the Pacific and was responsible for the design of a variety of modern structures. Later with the San Francisco office of Sverdrup and Parcel, consulting engineers, he directed the structural design of the multimillion dollar Albeni Falls and Dalles Hydroelectric Powerhouses.

As a senior design engineer of the International Engineering Co., Inc., a subsidiary of Morrison-Knudsen Co., Inc., Mr. Leontovich is now responsible for the design of many hydroelectric structures for clients in all parts of the world. He joined the Institute in 1955, and is also a member of ASCE.

Cassaro joins Prescon

Michael A. Cassaro has joined The Prescon Corp., Corpus Christi, Tex., as structural engineer. Mr. Cassaro, a graduate of Rensselaer Polytechnic Institute, formerly was city engineer in Troy, N. Y., heading the division responsible for design of roads, walls, maintenance, and other city projects.

Honor Roll

January 1—April 30, 1958

Membership in ACI offers a professional exchange of ideas, methods, and procedures dealing with concrete and concrete construction. It is through this exchange that each individual receives an ever increasing technical knowledge. To broaden this scope of knowledge, get your friends and acquaintances to join the American Concrete Institute.

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Bryon P. Weintz	2
Tsu-ming Yang	2

New Members

The Board of Direction approved 86 Individual applications, 4 Corporations, 23 Juniors, and 19 Students making a total of 132 new members. Considering losses due to deaths, resignations, and nonpayment of dues the total membership on Apr. 1, 1958, was 9424.

Individual

ADAMS, HAROLD, Fort Smith, Ark. (Arch.)
ALLAN, FRANK, Houston, Texas (Allan Ins. Agy.)
ALLEN, RICHARD G., San Leandro, Calif. (Sales Engr., Dewey & Almy Chemical Co.)
ANDREWS, JAMES B., Glenview, Ill. (Assoc. Devel. Engr., PCA)
ARNDT, HARRY, St. Petersburg, Fla. (Struct. Engr. for Transmission Dept., Florida Power Corp.)
BADGLEY, PAUL G., Syracuse, N. Y. (Monolithic Concrete Wall Co.)
BAIRD, ARTHUR M., Baldwin, N. Y. (Struct. Engr., Burns & Roe, Inc.)
BEARD, EARL G., Park Ridge, Ill. (Testing & Insp. Engr., Dept. of Pub. Wks., Bur. of Engrg., City of Chicago)
BIERBACH, EDWARD R., Denver, Colo. (Chf. Draftsman, Ketchum & Konkel)
BOOTH, GLENN C., Winnipeg, Man., Canada (Chf. Engr., Building Products & Coal Co., Ltd.)
BREMNER, ROBERT P., Milwaukee, Wis. (Vice-Pres., Industrial Div., The T. L. Smith Co.)
CHANG, SHAU-YEN, Olympia, Wash. (Jr. Bridge Engr., Design, Dept. of Hwy., State of Wash.)
COMBA, MRS. RACHEL P., Honolulu, Hawaii, (Mun. Librarian, City & County of Honolulu)
COSTON, OTIS D., Birmingham, Ala. (Mgr. & Partner, W. L. Coston & Sons)
DACENKO, MICHAEL, Winnipeg, Man., Canada (Struct. Engr., Design, Canadian Pacific Rlwy. Co.)
DAHLSTROM, TED, Chicago, Ill. (Asst. Sec., Home Modernization Inst.)
DAMBRUN, CARL D., Detroit, Mich. (Dist. Mgr., Pittsburgh Testing Lab.)
DAVIS, DONALD D., Ft. Worth, Texas (C. E. (GS-11) Design, Insp., Constr., Write Spec. for Air Navigation Facilities Bldgs., Civil Aeronautics Admin.)
DELGADO, JOSE A., Caracas, Venezuela (C. E.)
DI STASIO, JOSEPH, JR., Spring Lake Hghts., N. J (Struct. Engr., Partner, Di Stasio & Van Buren, Cons. Engrs.)
DONALD, JAMES L., Toronto, Ont., Canada (Struct. Sect. Ldr., Abitibi Power & Paper Co., Ltd.)
FEAD, J. W. N., Ft. Collins, Colo. (Assoc. Prof. of Civil Engrg., Colorado State Univ.)
FENNER, ROBERT D., Baton Rouge, La. (Struct. Engrg., Walter Kidde Engrs. S.W., Inc.)
FOLSOM, OLIVER H., New Dehli, India (Chf. Engr. Sect., Industry Div., International Cooperation Admin.)
FRANCONIA, WILLIAM P., Dorchester, Mass. (Struct. Lay-Out, also International Correspondence School, Groissier & Slager Iron Works & Concrete Columns)
GERMANIS, GUERRINO-RATKO-CARLO, Wellington, New Zealand (C. E., Ministry of Wks., Wellington Dist. Office, Arch. Div.)
GEYER, ROBERT I., Mt. Prospect, Ill. (Dist. Mgr. Sika Chemical Co.)
GUTTMAN, MARTIN, Camden, N. J. (Gen. Practice of Arch., Struct. Work)
HOLMSTEN, VICTOR T., Chicago, Ill. (Pres., Engrg., Exec., Consolidated Chimney Co.)
JARVI, ALBERT O., South Euclid, Ohio (Assoc., Trygve Hoff & Assocs.)
KHAN, SAMIULLAH, Karachi, Pakistan (Sr. Designer, MacDonald, Layton & Co., Ltd.)
KLEINE-KRACHT, J. A., Louisville, Ky. (Struct. Designer, Hartstern, Louis & Henry, Archs.)
KUNZE, WALTER E., JR., Evanston, Ill. (Asst. Mgr., Struct & Rlwys. Bur., PCA)
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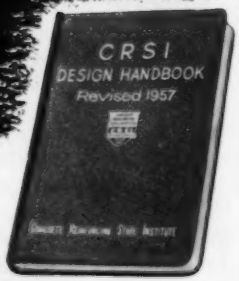

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MICKOS, GUSTAF, DRUMSO, Finland (Master of Engrg. Science, Chf. of Concrete Plant, RUDUS A. B.)
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ORAN, CENAP, Urbana, Ill. (Stud., Univ. of Ill.)
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SORLI, IVAR, Oslo, Norway (C. E., A/S Norsk Leca)
SOTERIADES, MICHAEL C., Athens, Greece (Proj. Engr., Iran, Asst. to Pres., Doxiadis Assocs.)
SWANSON, CLIFFORD L., Minneapolis, Minn. (Sec. & Public Relations, North Central Lightweight Aggregates Co., Inc.)
TAGGART, WILLIAM M., Los Angeles, Calif. (Struct. Engr.)
TASSIOS, T., Athens, Greece (Dr. of Engrg., Cons. Engr.)
TAYLOR, J. D., JR., Syracuse, N. Y. (Pres., Gen. Constr.)

ACI Concrete Primer

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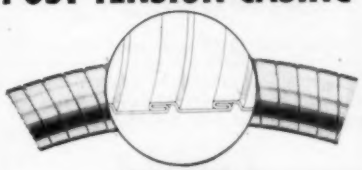

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Tools, Materials, Services

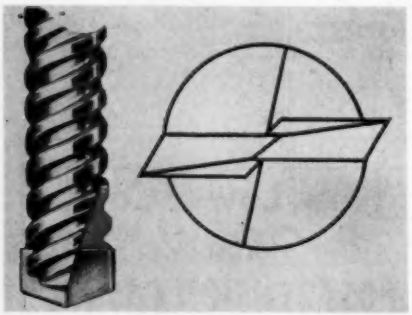
Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Concrete film for group showing

"Penny a Pound," a 16-mm motion picture describing the manufacture of cement, begins with the quarrying of limestone and shale and follows the rock on its trip through a modern and almost completely automatic cement plant. In full color with sound, this 20-min. educational film is available on free loan to any organization in northern and central California.—Calaveras Cement Co., 315 Montgomery St., San Francisco 4, Calif.

Interlocking cutter

An innovation for single-cutter masonry drills, an interlocking cutter is made of sintered tungsten carbide, and molded specifically for masonry drilling. The



manufacturer states that this cutter insures a stronger drill which will withstand heat that melts silver solder used in conventional drills. The need for coolants is thus minimized. Available in sizes ¼ through ½ in. —Tilden Tool Mfg. Co., San Clemente, Calif.

Concrete pressure pipe

Revised edition of the booklet "The Story of Concrete Pressure Pipe" offers current information on concrete pipe for engineers, contractors, and public officials. Fully illustrated, this 24-page booklet tells the history of concrete in water lines and its application to today's water and sewage systems. Included are simplified illustrations describing the principle of prestressing, and photographs depicting the methods used to manufacture prestressed concrete cylinder pipe; other sections cover pipe installation, tapping, joints, and maintenance.

The catalog section gives up-to-date information on sizes, weights, and normal operating heads of each type of pipe used for water transmission and distribution lines, subaqueous intakes and outfalls, and sewage force mains.—Price Brothers Co., 1932 E. Monument Ave., Dayton 1, Ohio

Anchor bolt fitting

The Deco anchor bolt fitting is said to completely eliminate the protruding anchor bolt that has long been a traffic hazard in new plant construction, and to eliminate the costly and time-consuming errors so often found in the initial setting of anchor bolts when concrete is being poured.

This anchor is flush with the floor until the actual installation of equipment or columns is made. The box-like fitting attached to the top of the anchor bolt contains an oblong nut, movable in all directions to compensate for the usual error in setting anchor bolts. The machine or column is attached to the fitting by use of a stud of the desired length, thus eliminating the hoisting and lowering of machines over a fixed bolt by the trial-and-error method.

The anchors are available as fittings only or complete with anchor bolt and stud to specifications in sizes from ⅝ to 1½-in. bolt diameter in ¼-in. increments.—Decatur Engineering Co., Dept. A, 519 E. William St., Decatur, Ill.

Admixtures catalog

On-the-job photos augment detailed data describing the use, properties, and advantages of various products including concrete admixtures Plastiment, retarding densifier, and Sikacrete, an accelerating densifier. Rugasol-C, a retardant concrete coating, and Rugasol-F, retardant form coating, are also covered. Igas joint sealer, nonmeltable mastic waterstop, and other coatings, impregnations, and additives are described. —Sika Chemical Corp., 29-49 Gregory Ave., Passaic, N. J.

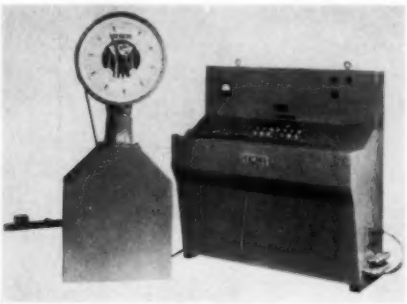
Front-mounted power loader

A front-mounted power loader for use on International 330, 300, and 350 utility tractors is designated the McCormick U-34B power loader. It can be powered either by the tractor's hydraulic system or with a special large capacity front-mounted hydraulic pump. A variety of interchangeable attachments adapts the loader to many loading, lifting, grading,

and material-moving jobs. Loader is available with either manual or hydraulic control of bucket.—International Harvester Co., 180 N. Michigan Ave., Chicago 1, Ill.

Portable batching plant control

Automatic cement weight cut-off controlled by a preset dial is one of several features of the master console pictured below. The control center is distribution



point for all electric, air, and hydraulic power required in a batching plant. Mixers, gates, conveyors, and pumps are controlled by pushbuttons and valves within arm's length of the operator.

Field wiring consists of connecting only incoming power to the main circuit breaker provided in the console. Lighting distribution circuits, solenoid valves, motors, and air and hydraulic supply lines are quickly connected and disconnected with positive locking receptacles. The manufacturer further states that the unit, constructed of heavy gage steel, will withstand the conditions of transporting and the rugged environment typical of cement batching plants. Complete control systems for portable or permanent batching operations are designed to customer specifications.—Gemco Electric Co., 25685 W. Eight Mile Road, Detroit 40, Mich.

Ready-mixed concrete trucks

Four-page brochure, giving basic specifications and components of a complete line of six-wheel-drive trucks for ready-mixed concrete service is now available. Data are given for mixer capacities from 5 to 8 cu yd.; wheel bases range from 160 to 190 in. The FWD trucks described have power-proportioned six-wheel drive, which delivers engine power to the front axle and both axles of the rear tandem in direct proportion to weight distribution.—Four Wheel Drive Auto Co., Clintonville, Wis.

Diamond masonry bits

Two types of "oriented" diamond masonry bits for drilling stone, concrete, asphalt, ceramics, and glass have been introduced. The first type is set with stones partially exposed, and this bit can be reset or rebuilt for further drilling after it has been used. The second type has small diamond particles impregnated throughout the matrix, and the bit is run to destruction.

Manufacturer states these bits insure smooth, clean, round holes for the passage of conduit, water pipes, or ventilating ducts, eliminating costly patching. Available in diameters ranging from ¾ to 14 in., sizes smaller than ¾ in. are supplied in the impregnated type only. Standard sizes permit drilling of holes up to 4½ or 12 in. in depth; other lengths are also available.—Sprague & Henwood, Inc., Scranton 2, Pa.

Chemical resistant concrete

Chemfast concrete when applied as a topping over a portland cement concrete base protects against nearly every commercially or commonly processed chemical that ordinarily causes expensive floor replacements and loss of time according to the developer of the product. Full instructions and personal review of chemically resistant floor problems now available.—Truscon Laboratories, Caniff and Grand Trunk Railroad, Detroit, Mich.

Marble mix for plaster

A dry mix of portland cement and marble, labeled Seablue White Marble Mix, is a hard, stain resistant, water repellent plaster aggregate, according to the manufacturer. Formulated to produce an extremely hard white finish with a minimum of maintenance, it is recommended for swimming pools, interior or exterior walls, and all other masonry surfaces. Requiring only water, the mix is applied with a steel trowel in the conventional manner.—Paddock Seablue Pool Equipment, Paddock of Texas, Inc., P. O. Box 7071, Dallas, Tex.

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1957 list and index of American Standards

A 67-page booklet lists 1723 voluntary national standards approved by the American Standards Association. These standards were developed by national organizations concerned with standards formulation and working either under their own auspices or within the framework of the ASA. There are 465 new and revised standards which were not listed in the 1956 edition of "List and Index of American Standards."

Booklet gives information on how to purchase American Standards, discounts and prices on special series such as civil engineering and construction standards.—Dept. DD-7, American Standards Association, 70 E. 45 St., New York 17, N. Y.

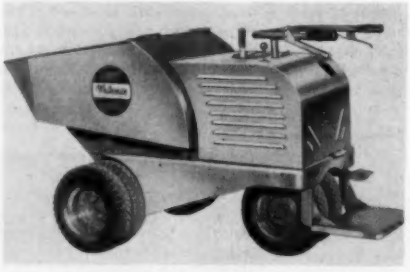
Viscometer for cement industry

The Cement Plastograph for use in controlling the viscosity of cement slurries in the wet-process cement industry is installed in such a manner that only a part of the main stream of the slurry is measured continuously. The slurry is reunited with the main stream after it has flowed through the sensing element.

The resistance encountered by the sensing element rotating in the slurry is expressed in opposite torque on the movable housing of the motor. This motion is transferred to a balance system by means of an arm fixed to the housing. The higher the torque is, in other words the higher the resistance exercised by the slurry on the sensing element, the greater will be the linear deflection of the lever. The balance system is connected with a small rugged torque scale which in turn converts torque into an electrical signal for actuating a large wall indicator or a strip chart recorder.—C. W. Brabender Instruments, Inc., 50 E. Wesley St., South Hackensack, N. J.

Walk-or-ride power buggy

The operator can either walk behind or ride on the fold-down platform of a power buggy which carries up to 10 cu ft of concrete. Powered by a Wisconsin



engine, it has front-wheel chain drive, automatic clutch, forward and reverse drive, and positive-control dumping mechanism. Speed is up to 5 mph. Manufacturer states this model is compact, lightweight and extremely maneuverable, designed to travel over narrow, light runways, onto elevators, and through doorways as narrow as 31 in.—Whiteman Mfg. Co., 13020 Pierce St., Pacoima, Calif.

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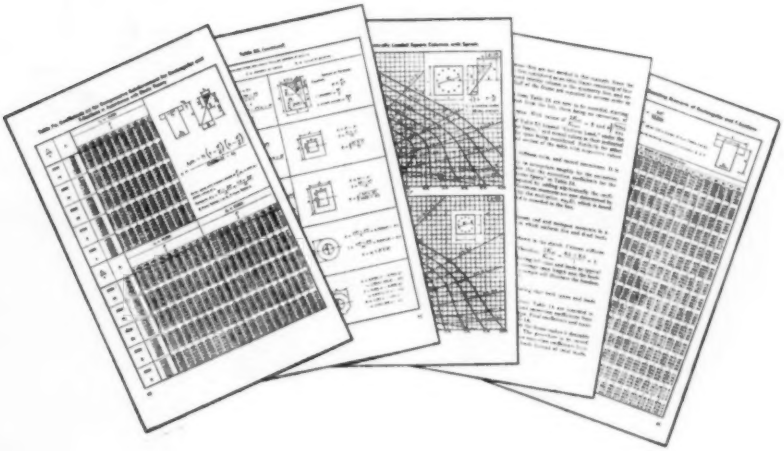

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